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US Army Corps
of Engineers
Los Angeles District

SANTA ANA RIVER BASIN, CALIFORNIA

Santa Ana River

Design Memorandum No. 1

PHASE II GDM ON THE SANTA ANA RIVER MAINSTEM including Santiago Creek

VOLUME 1
SEVEN OAKS DAM
APPENDIXES B THROUGH G

August 1988

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This volume accompanies the Main Report and the Supplemental Environmental Impact Statement for the Phase II General Design Memorandum for the Santa Ana River Mainstem including Santiago Creek and contains general design information relating to the Seven Oaks Outlet Works.

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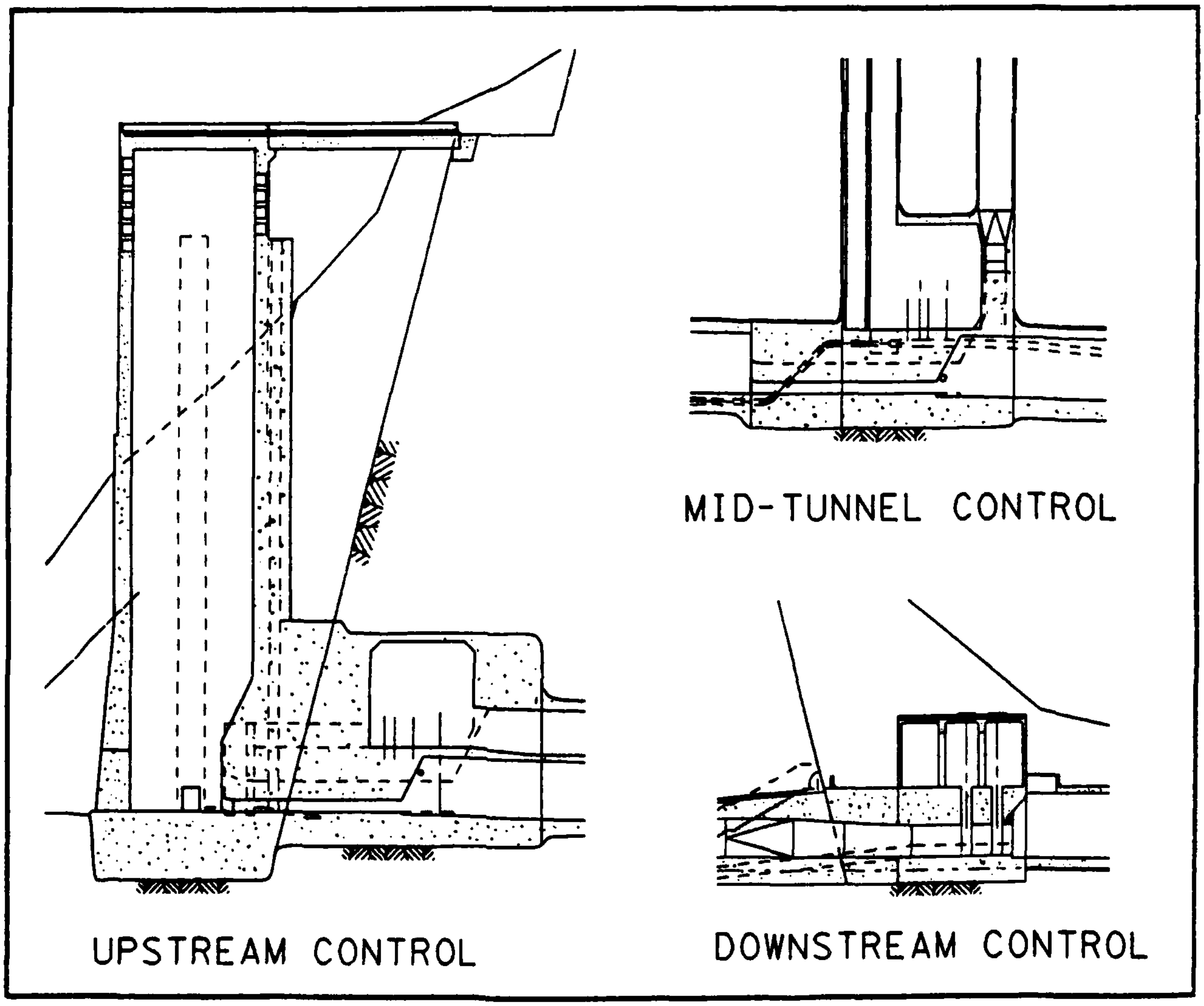
B OUTLET WORKS

US Army Corps
of Engineers
Portland District

SEVEN OAKS DAM OUTLET WORKS

PHASE II GENERAL DESIGN MEMORANDUM
FINAL

Prepared by Portland District
1 August 1988



SEVEN OAKS DAM OUTLET WORKS
GDM TEXT AS PREPARED BY PORTLAND DISTRICT
CORPS OF ENGINEERS, AUGUST 1988

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SECTION 1

OUTLET WORKS ALTERNATIVE INVESTIGATIONS AND COORDINATION

1.1 General. In November 1986, representatives of South Pacific Division (SPD), North Pacific Division (NPD), Los Angeles District (SPL), and Portland District (NPP) met in San Francisco, California, to discuss the Santa Ana Project and the possibility of design participation by Portland District. SPL asked that NPP consider general design memorandum (GDM) level design work for the following four project elements: Santiago Creek Drain, Oak Street Drain, Prado Dam, and Seven Oaks Dam. Subsequently, NPP agreed to perform the following specialized design in support of Phase II GDM level design: Seven Oaks Dam outlet works; Seven Oaks dam break analysis and overflow delineation; and Prado Dam outlet works. On 16 and 17 December 1986, SPL and NPP staff members met in Los Angeles to discuss NPP involvement in the Santa Ana Project and to visit the project sites. It was agreed that NPP would prepare conceptual designs for four alternatives for the regulating outlet works at Seven Oaks Dam. These alternatives were: upstream control with an inclined intake, central control with a vertical shaft, downstream control with a pressurized conduit, and upstream control with a horizontal gallery. It was also agreed that when the conceptual design evaluation was presented a decision would be made on which alternative to carry forward into more detailed design. The tentative plan was that NPP would perform the complete design for the intake structure upstream of the gates and/or transition section. The Memorandum of Understanding (MOU) and Scopes of Work (SOW) were negotiated, finalized, and agreed upon in the following weeks and the final MOU was signed between 5 February 1987 and 3 March 1987 by the appropriate offices.

1.2 Conceptual Design Meeting. On 2 March 1987, representatives of the technical staffs from SPL and NPP met in Los Angeles to review various conceptual designs developed by NPP for the outlet works at Seven Oaks Dam. On 3 March 1987, a meeting with technical and management staffs from

Office of the Chief of Engineers (OCE), SPD, SPL, and NPP was conducted to present the conceptual alternatives, review and comment on those alternatives, and concur upon a preferred concept.

a. Concepts and Alternatives. NPP investigated four basic concepts. In addition, several variables associated with the basic concepts were evaluated. A total of 13 alternatives were therefore studied, as shown in table 1-1.

Table 1-1. Estimated Costs (in millions of dollars)

Variable	I Inclined Access	II D/S Access	III Shaft	IV D/S Control
High level intake	$45	$32	$31	$28
Low level intake	45	30	29	26
High level transition			33	29
High head bulkhead			30	27
Steel-lined tunnel				30

b. General Design Assumptions. The following is a list of the initial general design assumptions used in the evaluation stage. Assumptions used in each technical specialty are discussed later.

(1) Intake trash openings - 2/3 (gate width).

(2) Diversion by staged construction through control section during dry season.

(3) Maintenance bulkheading required.

(4) Slide gates feasible, emergency gate required.

(5) Intake deck and access required in all cases.

(6) Concrete tunnel lining required for high velocities.

(7) Steel transitions downstream of control.

(8) Pressurized tunnel designed for applicable load combinations.

(9) Seismic defensive measures not considered.

(10) Downstream control options require steel lining.

c. Evaluation. NPP's presentation of the conceptual analysis included advantages, disadvantages, and costs (table 1-1). A brief summary follows:

(1) High level intake avoids sediment passage. The low level withdrawl system would require cleaning and maintenance and a vortex formation is possible.

(2) Low level intake has no vortex limitations. Sediment might require removal or enter conduits.

(3) Inclined access is more stable. Site conditions could greatly increase costs.

(4) Downstream access increases diversion capacity. Tunnel costs increase.

(5) Shaft access minimizes intake excavation. Difficult tunnel/shaft excavation and potential for earthquake damage. Complex construction and diversion sequence.

(6) Downstream control has best access to control. Pressurized tunnel would result in higher downstream velocities than desired.

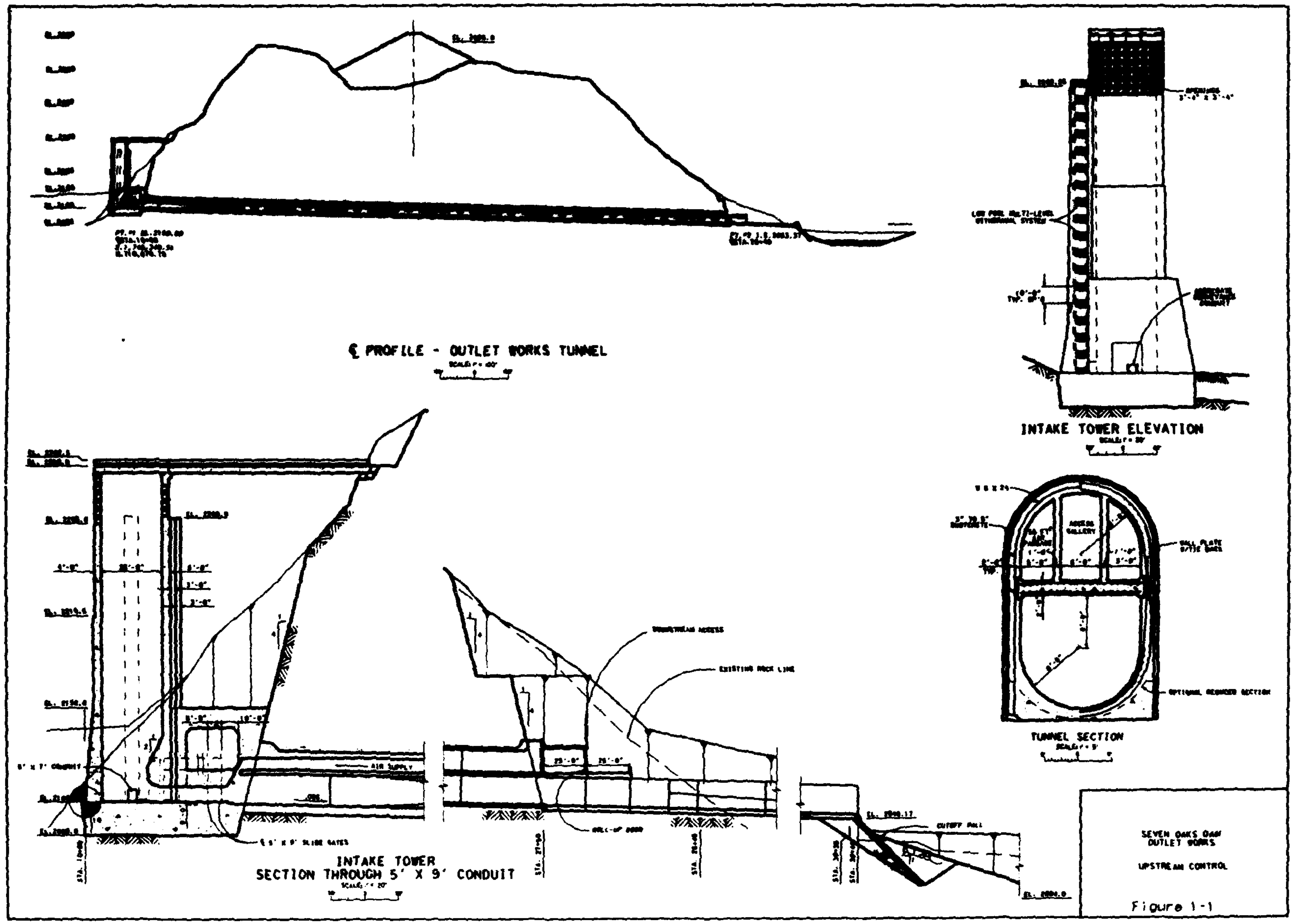
d. Issues. Several issues were identified during the meeting and the following is a list of the key issues for future consideration.

(1) The need for modeling at the GDM stage.
(2) Impact of an earthquake-induced displacement.
(3) Sediment deposition potential.
(4) Viability of a mid-tunnel control.

e. Recommendation. The initial recommendation for future design development was an alternative consisting of upstream control and downstream access (see figure 1-1). The primary advantages of this alternative were that it minimizes excavation activities, does not require a pressurized tunnel, provides the lowest downstream velocities, and offers the least impact from seismic events, relative to access. Agreement was reached to refine the upstream control intake tower concept, however, further studies were needed regarding a decision on a high or low intake.

1.3 Refinement of Alternative Analysis. From March through early June 1987, the outlet works design was accomplished with discussions (almost daily) with SPL to assure coordination on design issues. In April 1987, at the request of SPL, NPP expanded its scope of work to include the design of the outlet tunnel. Also, at about the same time, the estimate of sediment deposition against the reservoir embankment was increased significantly from 80 to 150 feet and the intake was redesigned to account for this increase. Provisions were made to allow multilevel reservoir withdrawal. NPP evaluated an option which would convey sediment through a conduit at the base of the intake structure. With the increased loads and height, NPP also evaluated a conventional tower configuration in addition to the gravity section which had been presented earlier.

1.4 On-Board Review Meeting. On 9 June 1987, representatives of the technical and management staffs from SPL and NPP met in Portland. The purpose of the meeting was to present and review the design status. In



℄ PROFILE - OUTLET WORKS TUNNEL
INTAKE TOWER ELEVATION
TUNNEL SECTION
INTAKE TOWER
SECTION THROUGH 5' X 9' CONDUIT
DOWNSTREAM ACCESS
EXISTING ROCK LINE
AIR SUPPLY
CUTOFF WALL
OPTIONAL REDUCED SECTION
ACCESS GALLERY
SEVEN OAKS DAM
OUTLET WORKS
UPSTREAM CONTROL
Figure 1-1

addition to design status, NPP presented updated design criteria and assumptions, cost estimates, and construction schedules. Summary of the meeting is generally as follows:

a. Status of Issues. A summary of the four previously identified key issues and the way in which each was resolved at that time are below.

(1) Need for Modeling. Concurrence was reached on modeling of the outlet works at the feature design memorandum (FDM) stage.

(2) Impact of 4-Foot Earthquake-Induced Displacement. The postulated 4-foot displacement of the outlet works is not expected to cause catastrophic damage or release of reservoir. Tunnels have an excellent performance record under seismic loadings. Defensive measures will be considered only where major shear/fault zones are encountered along the tunnel alignment. A subsequent 28 July 1987 letter of endorsement was prepared by CENPP-EN, subject, Seven Oaks Dam Outlet Tunnel, Seismic Criteria.

(3) Sediment Deposition Potential. Agreement had been reached earlier to provide for 150 feet of sediment deposition.

(4) Viability of a Mid-Tunnel Control (With Downstream Access) Alternative. Evaluation of this alternative with revised sediment height and tunnel design assumptions was made with costs of the two alternatives (upstream control versus mid-tunnel control) essentially the same. The upstream alternative was chosen because it offered: dependable diversion capacity; an effective construction sequence; and a less complex, more flexible operating system.

b. Discussions and Actions Required. Several issues were discussed following formal presentation and informal technical counterpart meetings. The following is a list of the major actions required as a result of those discussions.

(1) SPL to provide direction on the need for low-flow bulkheading.

(2) NPP to consider expanding scope of work to include structural design of outlet structure downstream of the tunnel portal.

(3) SPL to develop radio control plan.

(4) SPL to provide guidance on liquifaction potential around the tower.

(5) SPL to provide revised operation schedule.

(6) SPL and NPP to develop action plan for transition of involvement with Waterways Experiment Station (WES).

(7) SPL to provide design criteria for hydrostatic loading assumptions for the tunnel. (The actions taken have been included in this report.)

1.5 Significant Design Criteria Changes. Based on optimization studies performed by SPL following the June 1987 meeting, the following changes were made in October 1987.

a. Reservoir storage volume was reduced from 160,000 to 147,000 acre-feet.

b. Spillway crest elevation was changed from 2,598 to 2,580 feet, NGVD.

c. Sediment depth near the intake tower increased to 165 feet, or El. 2,265 instead of the previously assumed elevation of 2,250.

1.6 Downstream Control. In July 1987, SPL requested that Portland District reexamine the possibility of an outlet works that would feature a downstream flow control. This request came from a reconsideration of the need for steel lining with downstream control alternatives. Earlier

downstream control conceptual studies (table 1-1) indicated an appreciable savings was possible if a steel lining wasn't required. With the re-introduction of the downstream control, Portland District presented the concept of a steel conduit within the tunnel along with further studies of lined and unlined alternatives. SPL subsequently withdrew support of a downstream control with an unlined pressurized tunnel, citing geotechnical/dam safety concerns. Only steel lined downstream control alternatives were studied in detail and results are summarized as follows.

a. Preliminary Design. On 30 September 1987, an In-Progress Review meeting was held at the Los Angeles District Office relative to downstream control. At that time preliminary data was provided to SPL which compared two potential downstream control alternatives with the previously developed upstream control option. The two downstream control alternatives consisted of the steel pipe-within-a-tunnel concept and a full-size steel and reinforced concrete lined tunnel. At that time the cost estimates for the two alternative downstream control options were very close and both were estimated to have approximately $3 million less cost than the upstream control option. In addition to the cost estimate and layout details, the package also contained preliminary advantages and disadvantages for each of the downstream control alternatives.

b. Selection of Alternatives. Through several exchanges of conversations bewteen SPL and NPP technical staff representatives in late September and early October, NPP concentrated their efforts on the pipe within the tunnel concept for the downstream control option. In a letter to SPL dated 14 Oct, NPP indicated preliminary recommendation in favor of the downstream control option featuring the pipe within the tunnel instead of our previously recommended upstream control option. The basis for this recommendation was a lesser cost and several design advantages that the downstream control option included compared to the upstream control option. In a letter from SPL dated 22 October 1987, NPP was instructed to concentrate future studies for the downstream control option on the steel pipe within a tunnel concept rather than the steel lined concrete tunnel option. The reasons presented for preferring this alternative were as follows:

(1) The redundancy provided by a pipe within a tunnel better addresses the dam safety concerns for pressurized conduit in light of the 4-foot displacement criteria.

(2) The increased capability to survive eathquake displacements.

(3) The lower cost to repair the conduit following a displacement.

c. Technical Review Meeting on Downstream Control. On 19 November 1987, representatives of the NPP, SPL, SPD, and OCE held a check point meeting to further discuss the prelimiary design of the pipe within the tunnel alternative for downstream control. Advantages of the downstream control option were verified as follows:

(1) Least cost.
(2) Minimizes tower construction.
(3) Minimizes tunnel construction.
(4) Improved control access.
(5) Overbuild for seismic displacement.
(6) Inspection and maintenance of tunnel possible.
(7) Minimal use of 11-foot pipe (low flow bypass).
(8) Site change claims less likely.
(9) Built-in diversion with low flow system (for inspection and maintenance).
(10) Electrical and mechanical to upstream control eliminated.
(11) Smaller tower socked into rock. Moves point of fixity. Decreased seismic loading on tower.
(12) Temporary diversion liner can be allowed to relieve itself of external long-term hydrostatic pressures.
(13) Future power capability.

Some disadvantages of downstream control are as follows:

(1) Pipe maintenance costs will be equivalent to approximately $13,000 per year for painting.

(2) Complex diversion scheme.

(3) Increased maintenance for low flow system.

(4) Tight space for pipe installation and maintenance inside tunnel.

(5) Two contracts for tunnel/pipe completion.

(6) High downstream velocities (160 fps versus 120 fps).

(7) Complex emergency bulkheading system (if required).

These advantages/disadvantages of downstream control can be compared to the following primary advantages of upstream control:

(1) Non-pressurized tunnel.

(2) Minimizes exit velocity.

The primary disadvantages of upstream control are as follows:

(1) Permanent reinforced concrete liner required.

(2) Maximizes tower and tunnel construction efforts.

(3) Large tunnel diameter and two bench tunnel construction.

(4) Complex and expensive to provide measures to accommodate displacements due to seismic activity, ie., major earthquakes.

(5) Cavitation potential.

(6) Tunnel growth due to increased air demand.

(7) Higher freestanding tower potentially infeasible due to seismicity of site.

Follow on action items were agreed as follows:

(1) Portland District would proceed with finalizing the GDM documents to reflect the recommended downstream control option featuring the pipe within the tunnel alternative. No further work by Portland District will be done on the upstream control option. The question was raised concerning the justification for two low flow outlet pipes instead of one (an explanation of the need for two low flow outlets was provided to SPL in an attachment to the minutes of this meeting and forwarded on 20 November 1987). Both the low flow and the minimum discharge lines are required to meet the operation schedule.

(2) The rationale behind recommending a pipe within the tunnel for downstream option would be included.

(3) SPL was to provide operational criteria and justification for the upstream bulkhead gate contained in the present design and located within the intake tower. The GDM documents reflect the design criteria, operational justification, and intent for the maintenance bulkhead gate required by the design.

1.7 Mid-Tunnel Control. As a result of Technical Review Conference (TRC) held in Los Angeles on 12, 13, and 14 January, the option of a mid-tunnel control for the Seven Oaks Outlet Works was determined to deserve further evaluation as part of the GDM process. Portland District was asked to expand the concept level mid-tunnel control design studies and incorporate the mid-tunnel control option as a alternative into the final GDM.

a. TRC, January 1988. A number of significant conclusions and comments resulted from this TRC. The following paragraphs present the most significant of these issues.

(1) There was considerable concern over the downstream option concerning the possibility of pipe rupture and subsequent damage created by the resulting water jet, if this happened near either end of the conduit.

(2) For the downstream control option, the possibility exists under a full pool scenario that excessive hydrostatic pressures could be transmitted to the downstream end where the rock confining stresses near the portal might be insufficient to prevent a complete blowout of the gate control structure.

(3) The downstream control option would require considerable bracing of the pipe and potentially a high level of precision in the pipe alignment to avoid the effects of the very high forces due to the high water velocity. If the pipe in the tunnel became deformed or offset, the excessively high forces of the high velocity water flow would probably break the pipe loose from the anchoring system.

(4) The cost of the downstream control valve must be added to the downstream control option.

(5) The downstream control option must also include provisions for assuring undrained rock conditions at the upstream end of the tunnel.

(6) The most important dam safety objectives were identified as the following: the ability to control the reservoir after a major earthquake; the capability to inspect the outlet facilities following a major earthquake; and access for repairs following a major earthquake.

(7) It was decided that hydraulic modeling efforts could begin before the final design selection is made.

(8) The evaluation of the three options will be based on the following factors: constructibility (location/access/construction sequence and tunnel size); cost; survivability (earthquake shaking and deformations); functionality (post earthquake tunnel access, tunnel use and gate use); dam safety (reservoir control tunnel inspection, and access and repair); and finally precedence.

(9) The major design objectives will be identified for each of the design disciplines: geotechnical, structural, and hydraulic design.

b. Revised Scope, GDM Outlet Works. It was determined at the TRC meeting that the scope of the GDM outlet works appendix will be expanded to include equivalent details of each of the three options studied by Portland District; i.e., upstream control, downstream control, and mid-tunnel control. The mid-tunnel control design will be developed to the similar extent as the upstream and downstream control for the final GDM document. The alternatives are compared using the aforementioned criteria listed in paragraph 1.7.a.(8). This comparison is presented in section 5, Comparison of Alternatives.

SECTION 2

UPSTREAM CONTROL - OUTLET WORKS

2.1 General. This section describes the regulating outlet (RO) features for an outlet works with control at the upstream portal. The outlet works for this dam are located on and through the left abutment (see General Plan, plate 2-1). The system is comprised of a high level intake tower with upstream control gates, a regulating outlet/diversion access tunnel, a downstream access and equipment structure, and an outlet channel connecting the tunnel portal and an energy dissipating plunge pool.

2.2 Intake.

a. General. The intake tower is a 222.5-foot-high tower founded at elevation (El.) 2,080 within dioritic rock formation. The tower height was set based on an expected sediment deposition over the project life of 165 feet, or from El. 2,100 to El. 2,265. The sill of the intake tower for floodflows (normal operating maximum of 8,000 cubic feet per second [cfs]) is located above the 2,265 elevation. The top 146.5 feet of the tower are essentially a light circular structure with a maintenance deck and access bridge at El. 2,299. The tower is considered cantilevered above El. 2,156 while below it is embedded within the rock mass to provide fixity for the tower above the rock line. This is required to reduce the magnitude of seismic stresses within the tower. Below El. 2,156, the tower is characterized by a more massive rectangular section which houses the RO entrance, bulkheading, gate room, and downstream air supply and access. Below El. 2,265, on the left side of the tower, is the multilevel withdrawal system. This system is a series of small diameter intakes used to regulate the lower debris pool. Outflow from the withdrawal system discharges into the large 36-foot-diameter tower wet well or the minimum discharge line. The tower is designed to operate submerged. The debris pool after 100 years of reservoir sedimentation or a flood event at year zero will see the tower submerged, while a standard project flood (SPF) will submerge the tower by approximately 280 feet.

b. Approach Channel. For the first operating years, the diversion approach channel will become the operating RO intake channel. The channel will be approximately 30 feet wide at the bottom, with side slopes of 1V on 2H. The elevation of the invert at the tower will be 2,100 rising back naturally to the river channel upstream of the dam site at an approximate channel length of 1,040 feet and at a 3 percent slope.

c. Access. Tower access to the maintenance deck is required to perform yearly inspection, maintenance, bulkheading, and debris handling. The tower will be accessible from a road off of the top of the embankment dam from the right abutment across the upstream dam face, to the left abutment where it is cut into rock, to a point immediately downstream of the tower. The final access leg is accomplished by a 60-foot span single-lane bridge to the El. 2,299 maintenance deck. The road is 4,800 feet long, sloped at an average of 6 percent, single lane (with turnouts), and paved. The road will be designed for project crane and dump truck operations. Longer bridges were investigated to minimize rock excavations, but preliminary analyses found associated pier heights to be prohibitive in this seismic environment. Other road alignments on the left abutment were found to have excessive length and rock excavation. There was also a Southern California Edison penstock crossing to consider.

d. High Level Intake. The primary intake is designed to pass the regulated higher flows in accordance with the operating criteria as depicted in section 6. Upwards of 8,000 cfs can be passed as a normal design flow through the 116 3-foot 4-inch-square openings covering an area from El. 2,265 to El. 2,292.5. The openings are separated on all sides by 1-foot 6-inch beams (trash struts). Openings extend 315 degrees around the circumference of the intake tower. Eight hundred square feet are required for entrance velocity conditions; an additional 489 square feet have been provided as a safety factor against plugging. Flow past the trash struts enters a 36-foot-diameter wet well. In the ceiling of the intake is an 8-foot-square access hatch, provided for inspection, bulkhead placement, cleanout, and any maintenance required.

e. Wet Well. Within the tower is a 36-foot-diameter circular wet well. It extends for the full tower height, some 195 feet. The walls of the tower are constructed on top of a 20-foot-thick massive rectangular base slab. The walls are mass concrete from the base at El. 2,100 up to El. 2,156. In plan view, the tower below El. 2,156 is rectangular on the outside. Above El. 2,156, the tower is circular with a 5-foot-thick wall reducing to 4 feet for the top 84.5 feet. Near the bottom of the tower the downstream face of the wet well flattens to form a vertical plane where the rectangular bell-mouth entrances to the outlet conduits are found (see plate 2-3). Within the upstream face of the tower at El. 2,100, is a 5-foot-diameter conduit which will be used to facilitate maintenance at the intake structure.

f. Regulating Outlets. The entrances to the outlet conduits are found at the bottom of the large wet well. There are three bell-mouth openings which reduce to two 5- by 9-foot and one 2- by 3 1/2-foot conduits. The smaller opening will have provisions at the face for maintenance bulkheading. The larger conduits have 3-foot by 7-foot bulkhead slots located 18 feet downstream of the bell-mouth entrance. One maintenance bulkhead would be stored in the slot. The bulkhead slots are required to extend upward to an elevation above the El. 2,265 expected sediment level. The bulkhead will be for maintenance only, requiring a slide-type gate to be used only under static conditions. Should the reservoir fill while the bulkhead gate is in use, a downstream filling system will be provided to equalize pressure for bulkhead removal.

g. Hydraulic Slidegates. The two 5- by 9-foot operating slide gates are located 65 feet downstream of the outlet conduit intakes. Emergency slide gates are located 8 feet upstream of the operation gates (see plates 2-3 through 2-8). Smaller 2-foot by 3 1/2-foot low flow gates are located as shown on plate 2-3. The low flow entrance will require a trashrack with maximum bar spacing of 16 inches. The 2-foot by 3 1/2-foot low flow gates are required in conjunction with the minimum discharge line. The low flow gates discharge lower range flows up to 600 cfs. The minimum discharge line releases flows of between 10 and 90 cfs. Immediately downstream of the operating gate, air is introduced by use of aeration offsets located about the perimeter of the conduit. This air is brought from downstream by passages constructed above the outlet conduit.

h. Gate Room. A 31-foot by 35-foot gate room with a 25-foot-high ceiling is provided to house the hydraulic slide gates, and mechanical and electrical operating equipment. Overhead hoists are provided for maintenance. The room is sized primarily for the handling of gate components and to house the aforementioned project equipment.

i. Multilevel Withdrawal System. The multilevel withdrawal system (MWS) consists of two columns of 2-foot 3-inch-diameter ports spaced at 10 vertical feet for the full height of the sediment range (El. 2,100 to El. 2,265). The ports are located on the left side of the tower and have horizontal column spacing of 4 feet 6 inches on centerline. This system regulates the reservoir storage area below the intake tower sill at El. 2,265. The MWS ports are covered with trashrack grating and discharge into an 8-foot by 8-foot 6-inch wet well. An access hatch for maintenance will be located at the top of the wet well. The wet well discharges into the larger 36-foot wet well through a 5- by 7-foot conduit at El. 2,100. A trashrack with 6-inch square openings will be provided between wells. A large manually operated slide gate will be located at the conduit entrance with an operator located above El. 2,265. This gate will not be used as a throttling gate, because it will be used either fully opened or fully closed. A stoplog slot is located between the trashracks and wet well. Prior to flood season, sediment stoplogs will be installed. The concrete stoplogs will be placed in advance of the rising sediment (approximately 20 to 30 feet above the sediment level to account for the predicted rise in sediment depth from an SPF event). With the stoplogs utilized, sediment passage through the project will be minimized.

j. Minimum Discharge Line. Flows between 0 and 90 cfs will be passed through a minimum discharge line (16-inch pipeline) originating at the bottom of the MWS wet well. All these flows cannot be accommodated by the hydraulic slide gates between pool Els. 2,110 and 2,350, see figures 6-6 and 6-7. An operating shaft with an operating and emergency valve will be provided with access from the gate room (see plate 2-4). The valves will have power-driven operators and will be controllable from the downstream access structure. An alternative design is being considered to provide a 3-foot-diameter pressure pipe to carry flow to the downstream end of the outlet works. Flow would be regulated at the downstream end by a cone valve.

k. Instrumentation. The tower will be typically tied into the project survey monitoring program with key points identified by embedded monuments in the tower concrete. Tiltmeters and extensometers will be utilized. A seismic accelerograph will be located in the gate room. Hydraulic instrumentation about the slide gates and immediately downstream will be utilized to monitor pressure and flow conditions. A specific plan and types of instrumentation should be developed at the FDM level.

2.3 Regulating Outlet Tunnel.

a. General. The outlet tunnel is 1,627 feet in length and can be constructed either as an oval or horseshoe utilizing conventional drill and blast techniques (see plate 2-2). Flow will be open channel. Above the channel is an adit for access to the tower from the downstream portal. Adjacent to the adit are the two air supply conduits sized to provide 5,200 ft^3/s. Each conduit is 50 square feet and has the capability to provide additional air at other points within the tunnel if required.

b. Upstream Transition. Once through the slide gate, the flow passes through offsets which expand each wall by 6 inches and the floor by 1 foot. For the next 60 feet splitter walls are provided to contain flow until the discharge jet comes in contact with the invert after leaving offsets. The flow discharges into a 24-foot-wide by 16-foot-high section and is constant for 20 feet. The rectangular channel floor then transitions over 150 feet to a 9-foot radius half circle. The remainder of the tunnel keeps this shape. The circular channel bottom was desired as a more efficient structural element to resist external hydrostatic loading.

c. Tunnel Proper. The majority of the tunnel, 1,430 feet, is excavated as either a horseshoe or oval tunnel (contractor option) with an 18-foot-wide by 16-foot-high outlet channel. The tunnel base slope is .026. Above the channel is an access gallery to the tower. The adit is 6 feet wide and 13 feet high. Electrical conduits, lighting, and HVAC duct work will be located in the ceiling. On either side of the gallery

are air supply conduits for the slide gate aeration offsets. The conduits are designed to meet expected air demands with additional capacity in the event more air is needed downstream within the tunnel. Each conduit supplies 50 square feet of air passage. The air passage and gallery exit through the downstream access structure.

2.4 Downstream Outlet Structures.

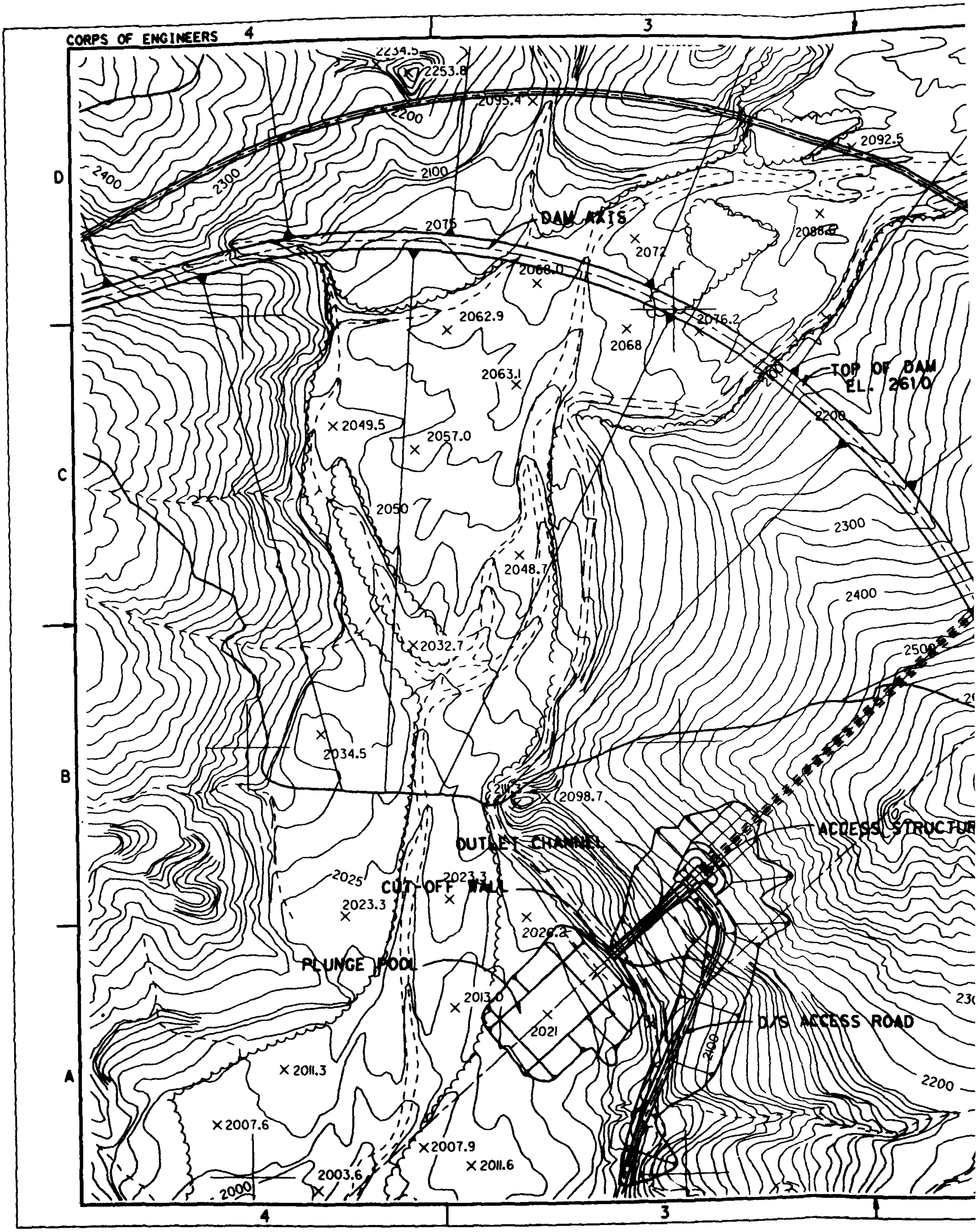
a. Access Structure. The downstream access structure has interior dimensions of 25 by 35 feet. This structure houses project equipment (diesel generator, reservoir readouts, control annunciation, HVAC, etc.), restroom, and storage, and is the upstream gate room access (see plate 2-6). A large rollup door is provided to bring out equipment components.

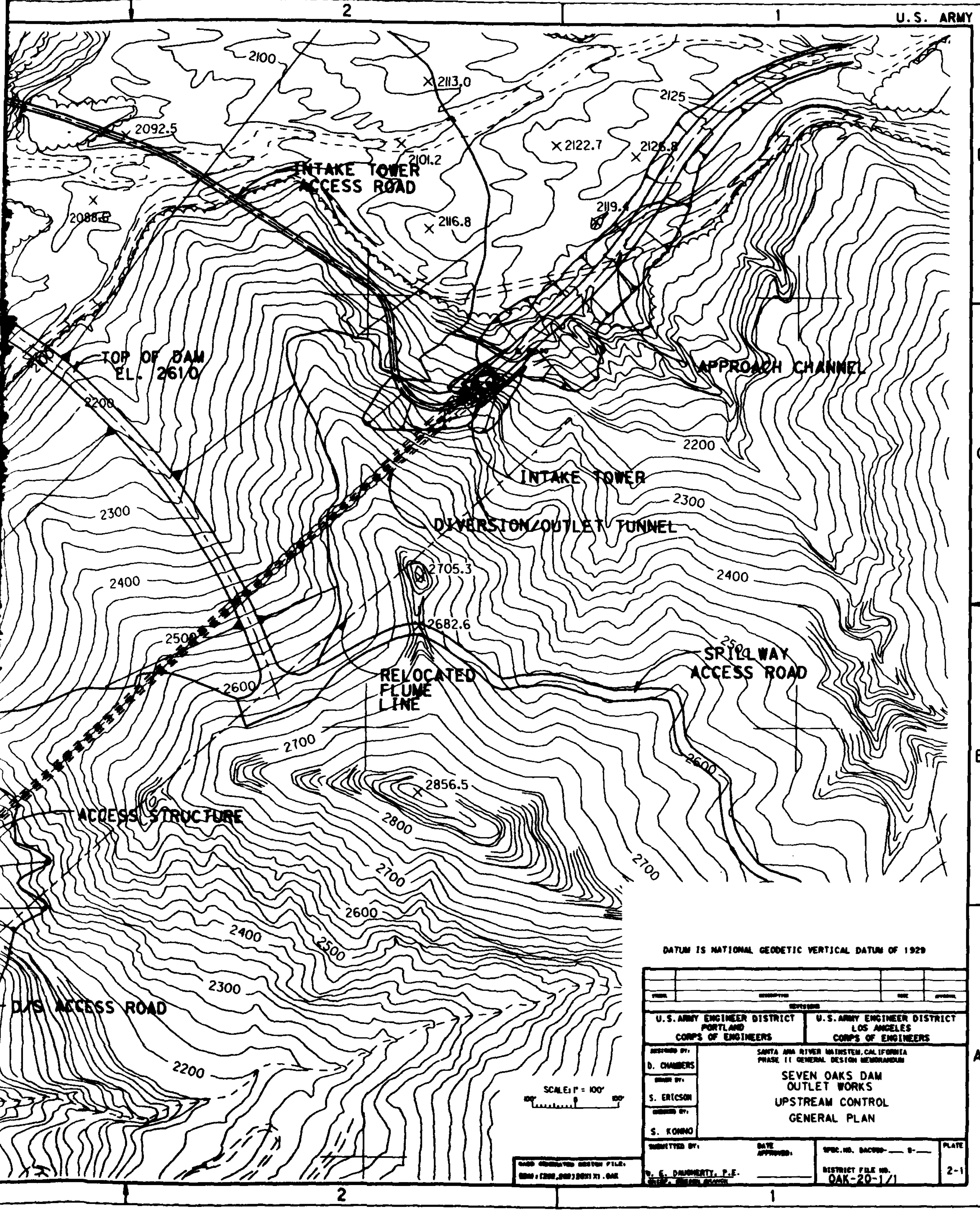
b. Outlet Channel. Exiting beneath the access structure is the outlet channel. This channel maintains its circular bottom cross section until just immediately downstream of the access deck where it begins a 190-foot transition back to a rectangular section prior to discharging into the plunge pool (see plate 2-6). The channel begins as a U-shaped wall founded on rock, but finishes out placed on a bedding of processed backfill.

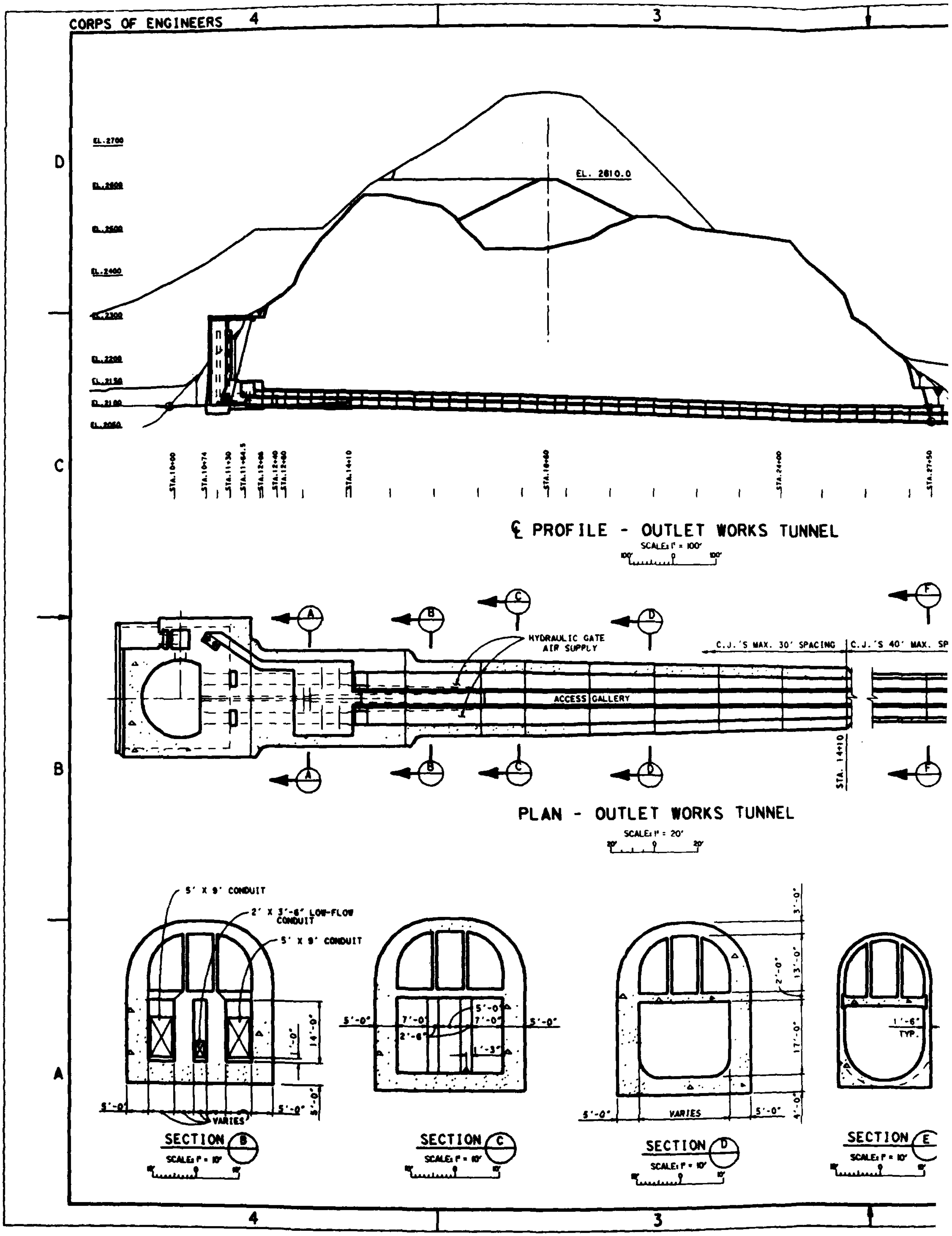
c. Cutoff Wall. The channel wall terminates 1,869 feet downstream of the operating slide gate. At this point a cutoff wall has been placed at the upstream edge of the plunge pool. The cutoff wall protects against upstream undercutting and is assumed to be required as a near vertical faced wall.

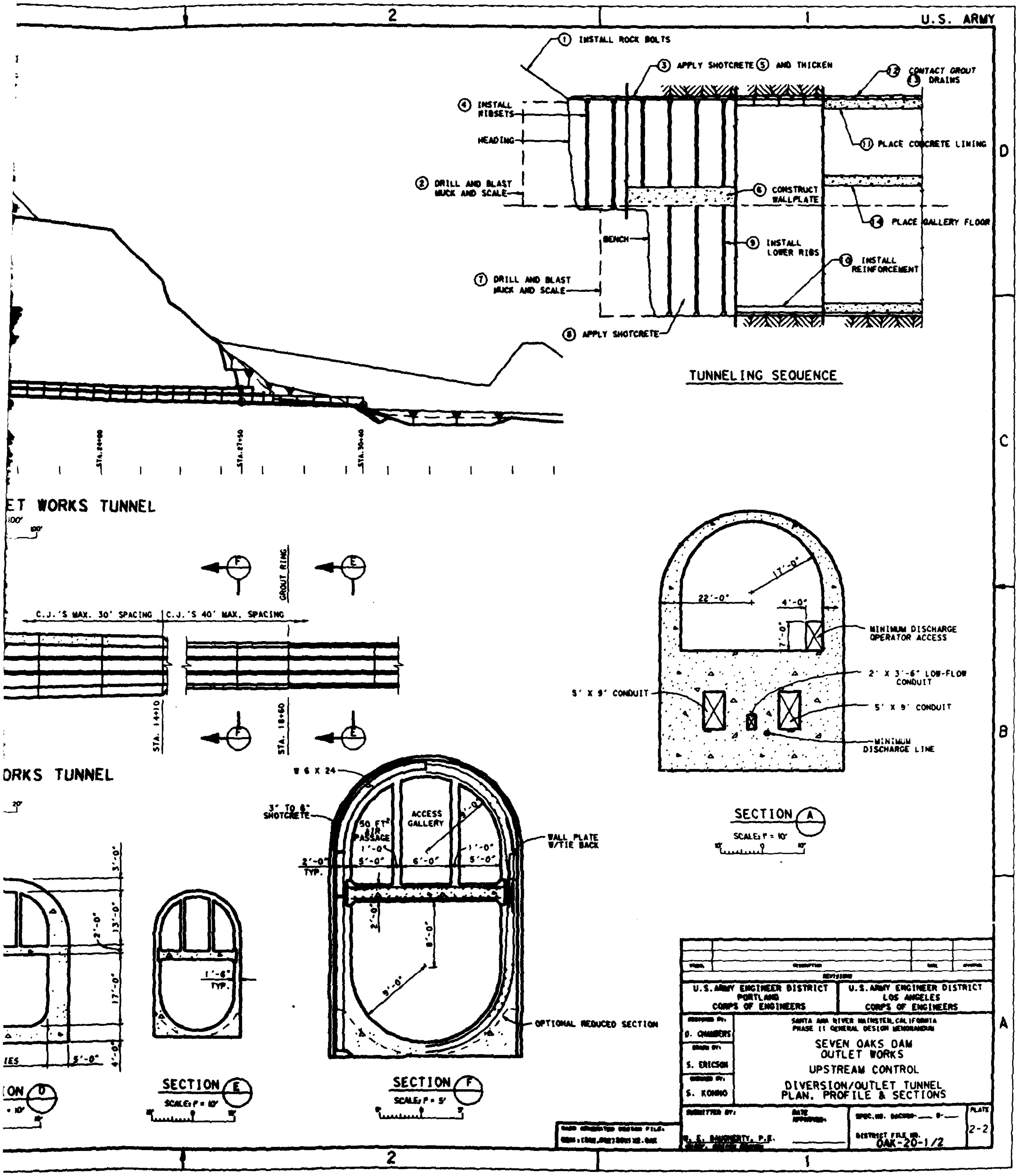
d. Plunge Pool. A scour hole will be pre-excavated to two-thirds the depth of the hole that, it is estimated, would form from a constant 8,000 cfs release. This depth will provide initial energy dissipation to protect structures in the downstream channel until the scour hole reaches its equilibrium depth. The estimated final scour hole is a sufficient distance downstream that it will not pose any danger to the dam embankment. It is 740 feet long and varies from 120 feet to 290 feet wide at the bottom with side slopes of 1V on 3H. Larger rock will be left in the basin to accelerate the armoring process.

e. Downstream Access Road. The downstream access road will be a single lane, paved road traversing the left abutment as shown (see general plan).

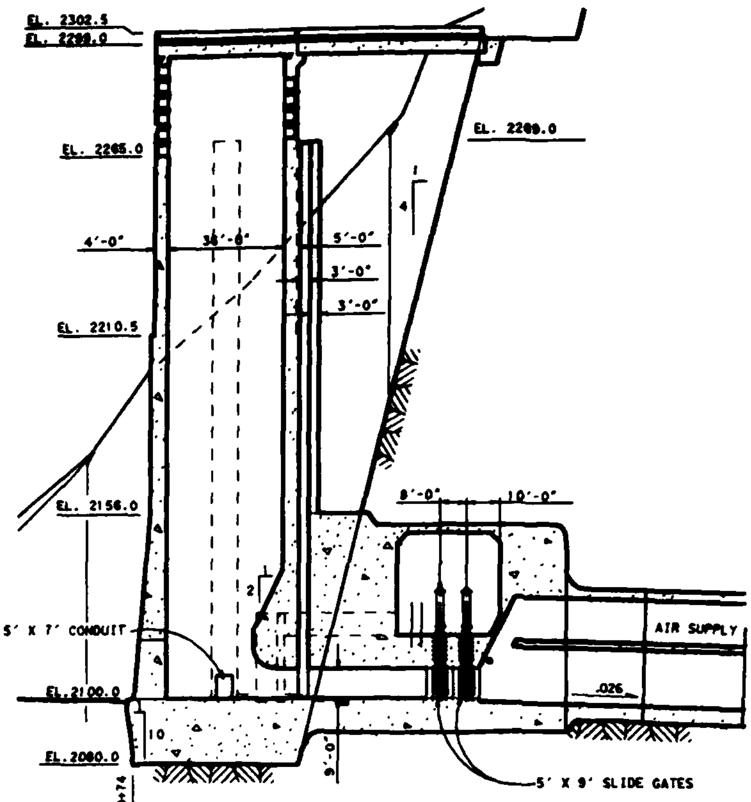

CORPS OF ENGINEERS
4
3
D
C
B
A
DAM AXIS
TOP OF DAM
EL. 2610
OUTLET CHANNEL
CUT-OFF WALL
PLUNGE POOL
ACCESS STRUCTURE
D/S ACCESS ROAD
2253.8
2095.4
2092.5
2088.0
2072
2068.0
2062.9
2076.2
2068
2063.1
2049.5
2057.0
2050
2048.7
2032.7
2034.5
2098.7
2023.3
2026.2
2013.0
2021
2011.3
2007.6
2007.9
2011.6
2003.6
2000
2025
2075
2100
2200
2300
2400
2500


U.S. ARMY
2100
2113.0
2125
2092.5
2101.2
2122.7
INTAKE TOWER ACCESS ROAD
2088.6
2116.8
2119.4
TOP OF DAM EL. 2610
APPROACH CHANNEL
2200
INTAKE TOWER
2300
DIVERSION/OUTLET TUNNEL
2705.3
2400
2682.6
2500
SPILLWAY ACCESS ROAD
RELOCATED FLUME LINE
2600
2700
2856.5
ACCESS STRUCTURE
2800
D/S ACCESS ROAD
SCALE: 1" = 100'
DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929
U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS
U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS
D. CHAMBERS
S. ERICSON
S. KONNO
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
OUTLET WORKS
UPSTREAM CONTROL
GENERAL PLAN
PLATE
2-1
DISTRICT FILE NO.
OAK-20-1/1

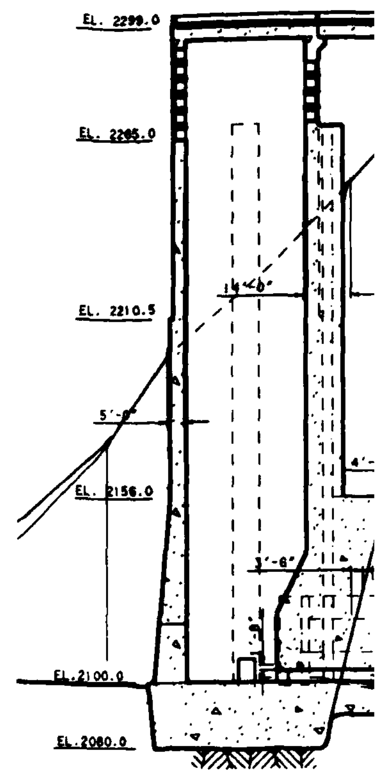

CORPS OF ENGINEERS
4
3
D
C
B
A
EL. 2700
EL. 2600
EL. 2500
EL. 2400
EL. 2300
EL. 2200
EL. 2150
EL. 2100
EL. 2050
EL. 2610.0
STA. 10+00
STA. 10+74
STA. 11+30
STA. 12+00
STA. 14+10
STA. 18+00
STA. 24+00
STA. 27+50
℄ PROFILE - OUTLET WORKS TUNNEL
SCALE: 1" = 100'
HYDRAULIC GATE AIR SUPPLY
C.J.'S MAX. 30' SPACING
C.J.'S 40' MAX. SP
ACCESS GALLERY
STA. 14+10
PLAN - OUTLET WORKS TUNNEL
SCALE: 1" = 20'
5' X 9' CONDUIT
2' X 3'-6" LOW-FLOW CONDUIT
5' X 9' CONDUIT
14'-0"
11'-0"
5'-0"
VARIES
SECTION B
SCALE: 1" = 10'
5'-0"
7'-0"
2'-6"
1'-3"
SECTION C
SCALE: 1" = 10'
3'-0"
13'-0"
2'-0"
17'-0"
4'-0"
VARIES
SECTION D
SCALE: 1" = 10'
1'-6" TYP.
SECTION E
SCALE: 1" = 10'


U.S. ARMY
(1) INSTALL ROCK BOLTS
(3) APPLY SHOTCRETE (5) AND THICKEN
(12) CONTACT GROUT
(13) DRAINS
(4) INSTALL RIBSETS
HEADING
(11) PLACE CONCRETE LINING
(2) DRILL AND BLAST MUCK AND SCALE
(6) CONSTRUCT WALLPLATE
(14) PLACE GALLERY FLOOR
BENCH
(9) INSTALL LOWER RIBS
(10) INSTALL REINFORCEMENT
(7) DRILL AND BLAST MUCK AND SCALE
(8) APPLY SHOTCRETE
TUNNELING SEQUENCE
STA. 24+00
STA. 27+50
STA. 30+40
ET WORKS TUNNEL
GROUT RING
C.J.'S MAX. 30' SPACING
C.J.'S 40' MAX. SPACING
STA. 14+10
STA. 18+60
ORKS TUNNEL
22'-0"
17'-0"
4'-0"
7'-0"
MINIMUM DISCHARGE OPERATOR ACCESS
2' X 3'-6" LOW-FLOW CONDUIT
5' X 9' CONDUIT
5' X 9' CONDUIT
MINIMUM DISCHARGE LINE
SECTION A
SCALE: 1" = 10'
W 6 X 24
3" TO 6" SHOTCRETE
50 FT² AIR PASSAGE
ACCESS GALLERY
WALL PLATE W/TIE BACK
2'-0" TYP.
1'-0"
5'-0"
6'-0"
8'-0"
9'-0"
1'-6" TYP.
3'-0"
13'-0"
17'-0"
4'-0"
OPTIONAL REDUCED SECTION
SECTION E
SCALE: 1" = 10'
SECTION F
SCALE: 1" = 5'
REVISIONS
U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS
U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS
D. CHAMBERS
S. ERICSON
S. KONNO
SANTA ANA RIVER MAINSTEM, CALIFORNIA PHASE II GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM OUTLET WORKS
UPSTREAM CONTROL
DIVERSION/OUTLET TUNNEL PLAN, PROFILE & SECTIONS
SUBMITTED BY:
DATE APPROVED:
PLATE
2-2
DISTRICT FILE NO.
OAK-20-1/2

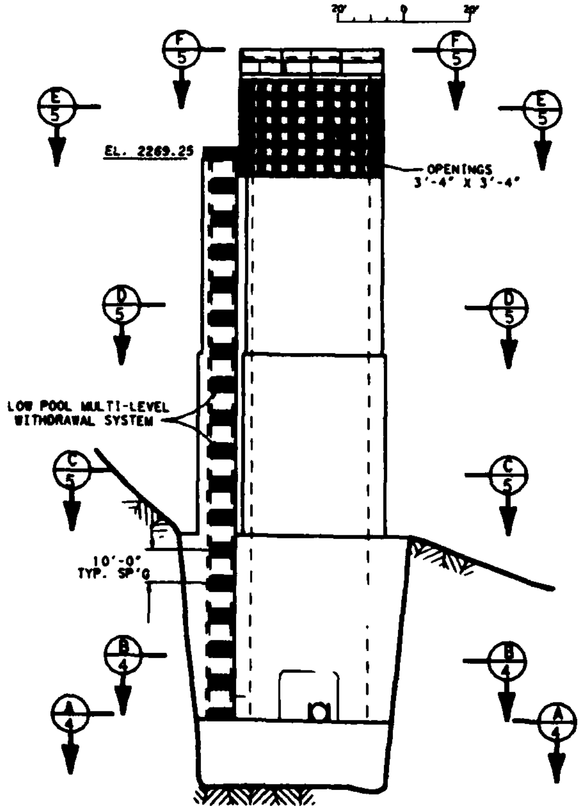
CORPS OF ENGINEERS



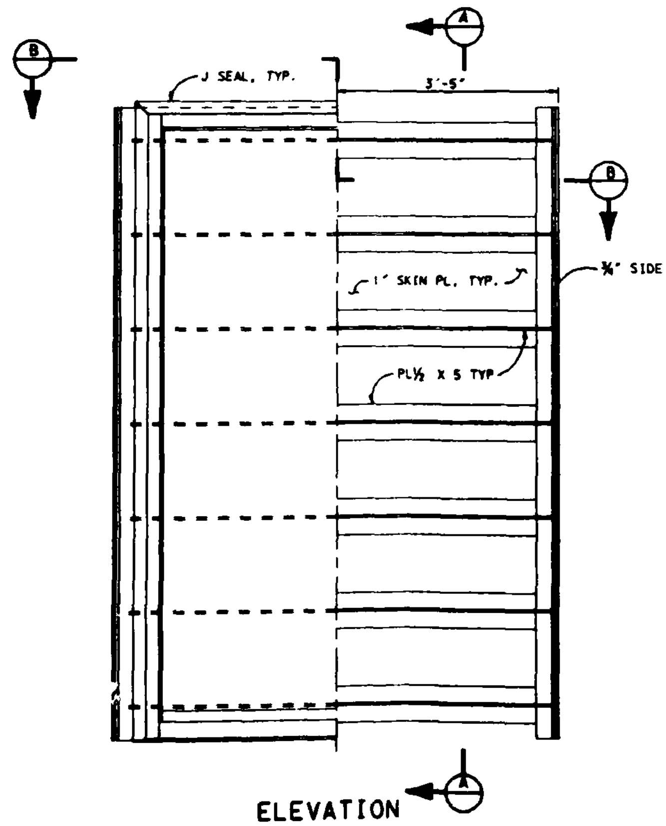
INTAKE TOWER
SECTION THROUGH 5' X 9' CONDUIT
SCALE: 1" = 20'



SE



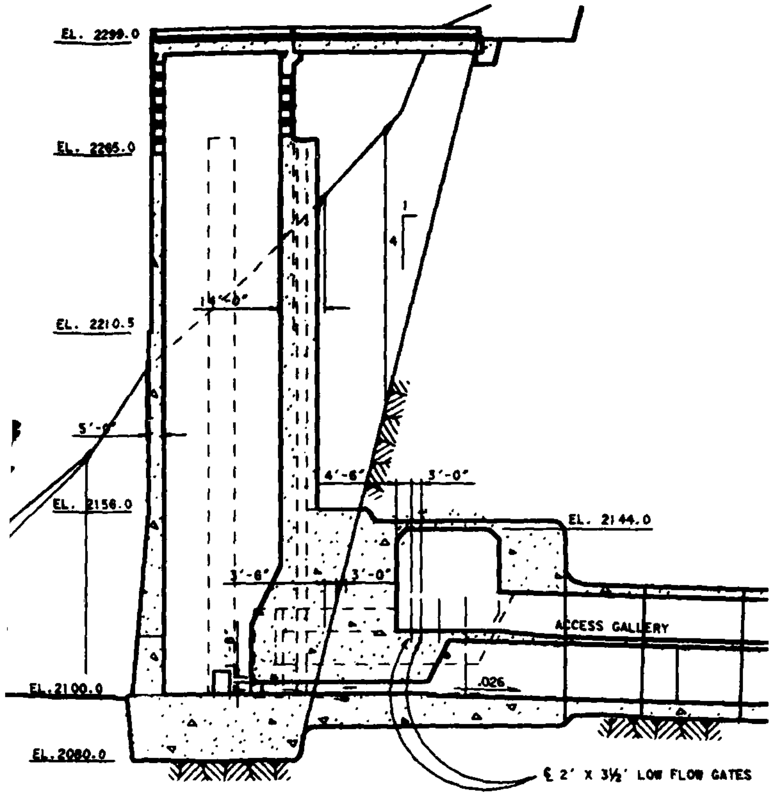
INTAKE TOWER ELEVATION
SCALE: 1" = 20'



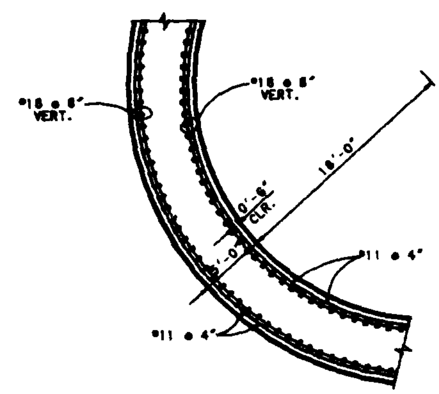
ELEVATION

R.O. MAINTENANC

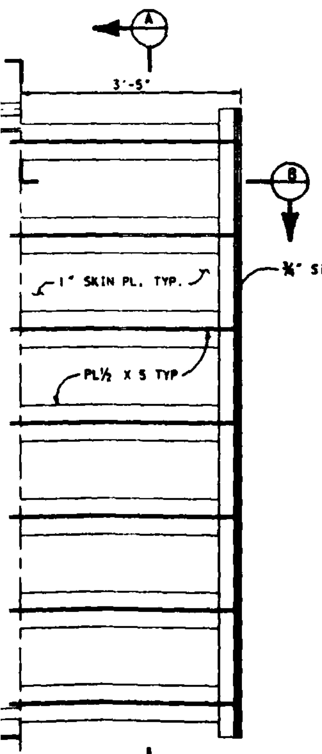
U.S. ARMY

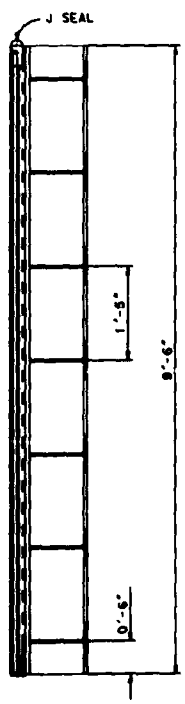


INTAKE TOWER
SECTION THROUGH ℄
SCALE: 1" = 20'

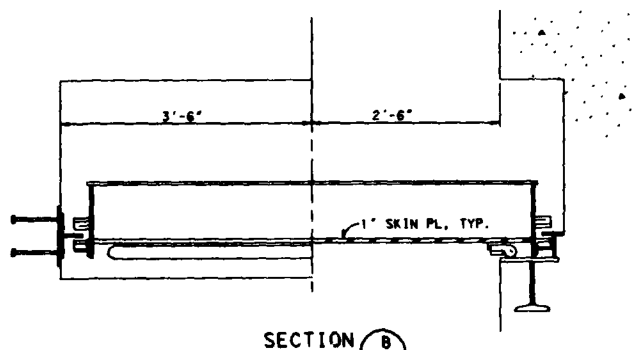


TOWER REINFORCEMENT (TYP.)
SCALE: 1" = 5'





SECTION A



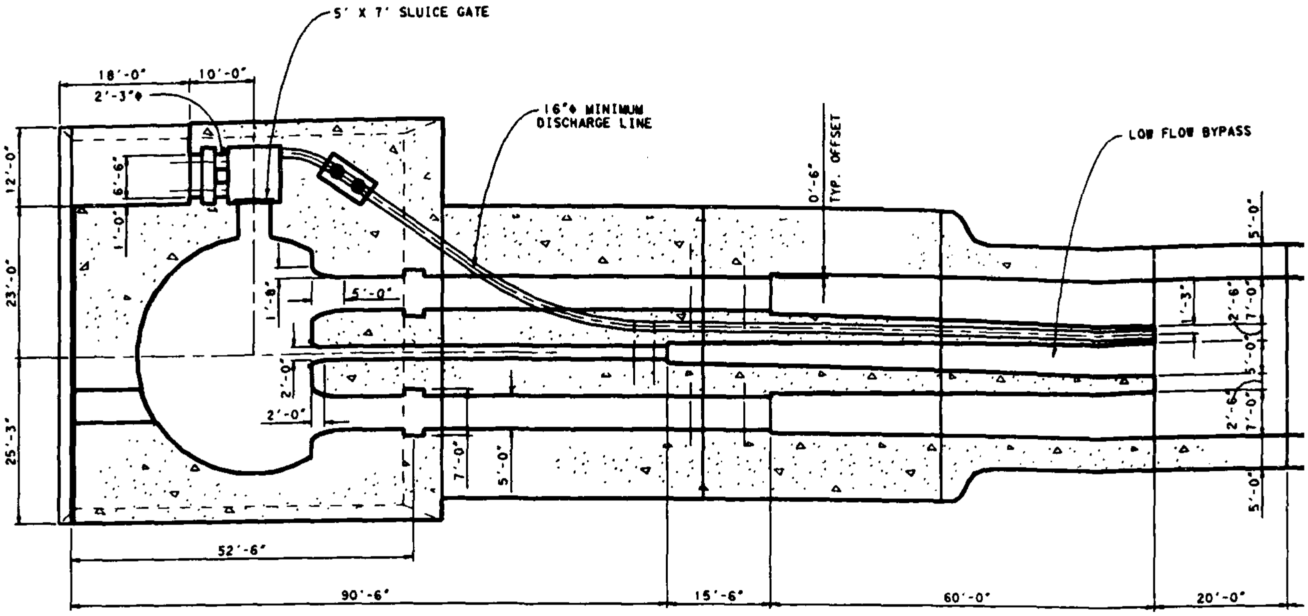
SECTION B
PLAN - R.O. BULKHEAD

ATION
R.O. MAINTENANCE BULKHEAD

U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS	U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS
DESIGNED BY: D. CHAMBERS	SANTA ANA RIVER MAINSTEM, CALIFORNIA PHASE II GENERAL DESIGN MEMORANDUM
DRAWN BY: S. ERICSON	SEVEN OAKS DAM OUTLET WORKS UPSTREAM CONTROL
CHECKED BY: S. KONNO	INTAKE TOWER SECTIONS, ELEVATION & MISC. DETAILS
SUBMITTED BY: R. E. DAUGHERTY, P.E.	DISTRICT FILE NO. OAK-20-1/3
PLATE	2-3

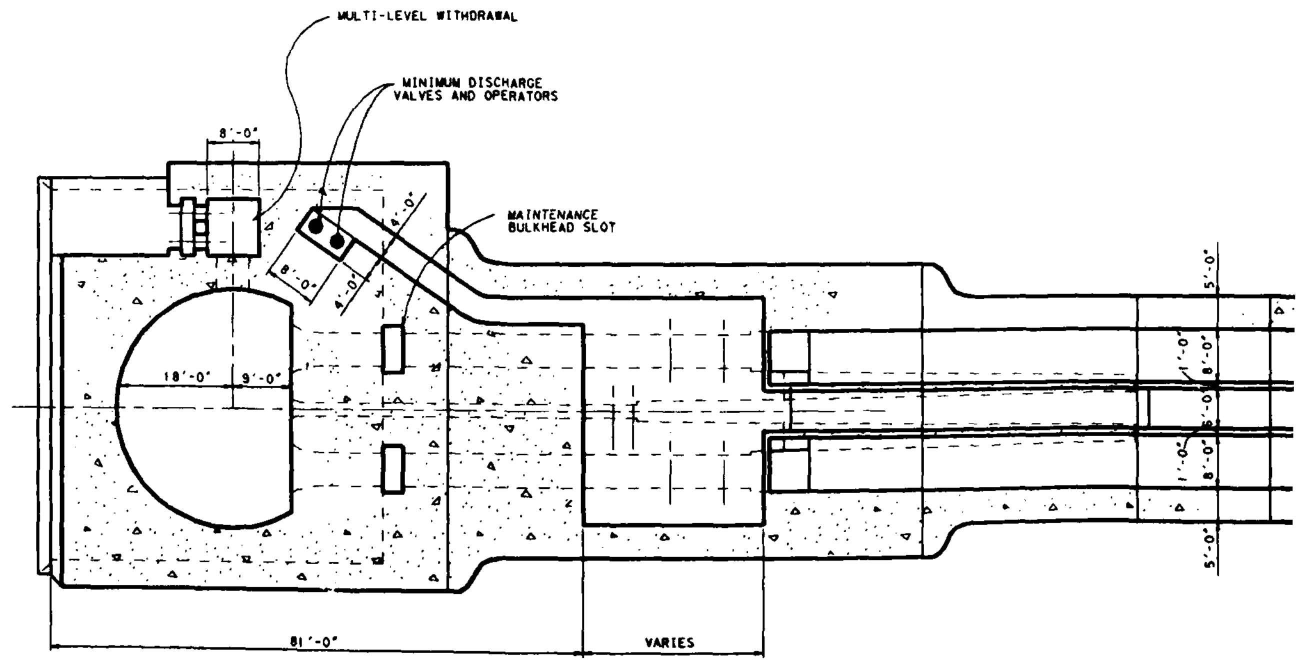
CORPS OF ENGINEERS

4 | 3



SECTION (A / 1-3)

SCALE: 1" = 10'

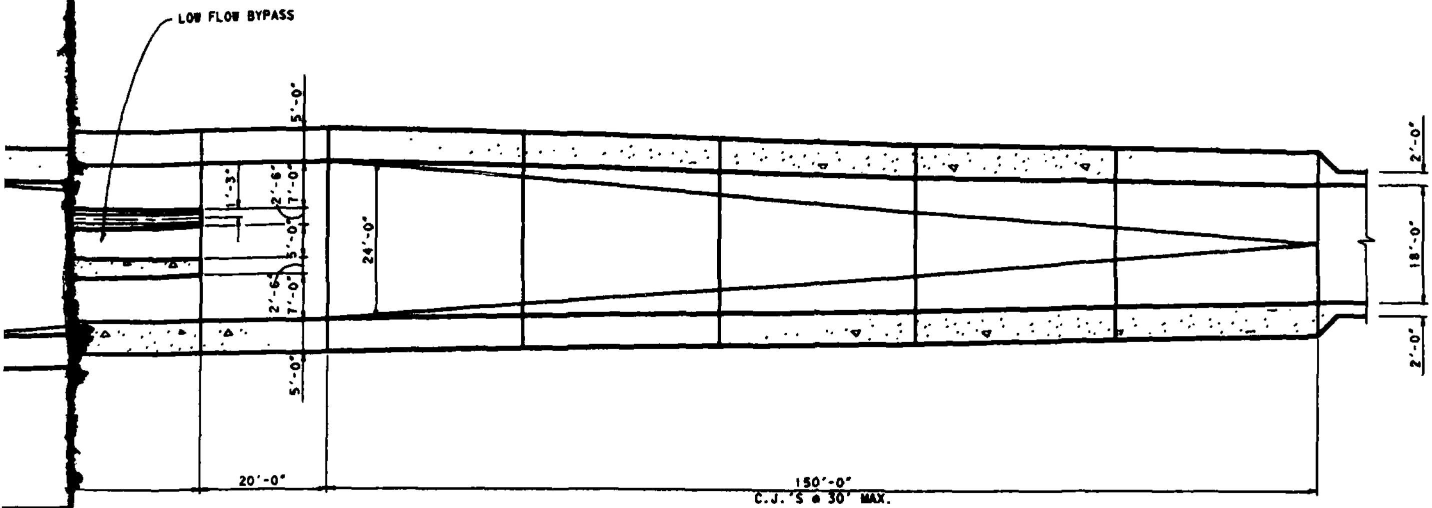


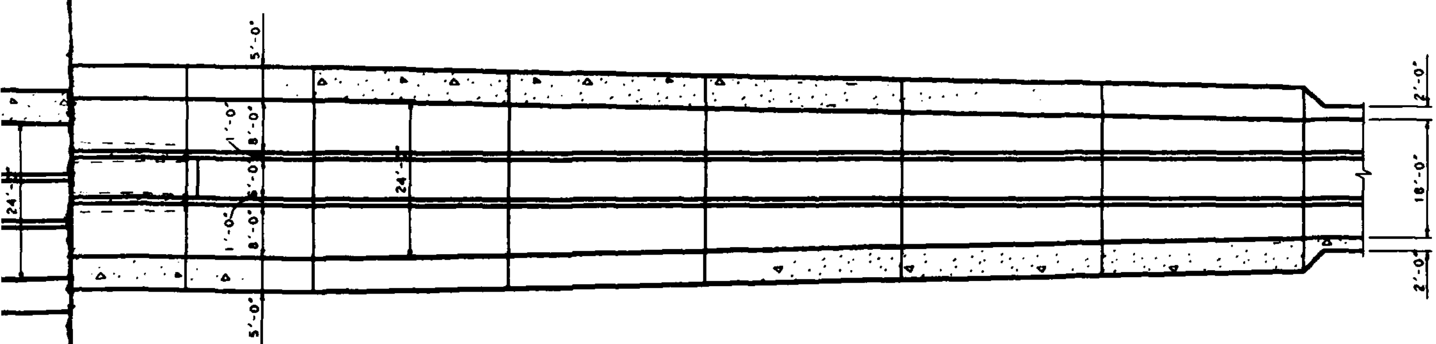
SECTION (B / 1-3)

SCALE: 1" = 10'

4 | 3

U.S. ARMY





REVISIONS

U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS	U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS

DESIGNED BY: D. CHAMBERS

DRAWN BY: S. ERICSON

CHECKED BY: S. KONNO

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
OUTLET WORKS
UPSTREAM CONTROL
INTAKE TOWER
PLAN SECTIONS SHEET 1

SUBMITTED BY:

DATE APPROVED:

DAUGHERTY, P.E.

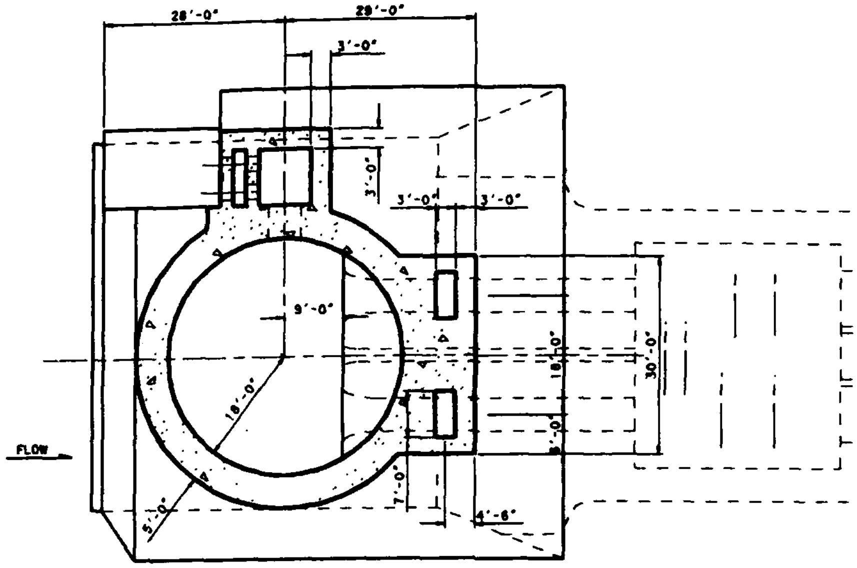
SPEC. NO. DACW09-

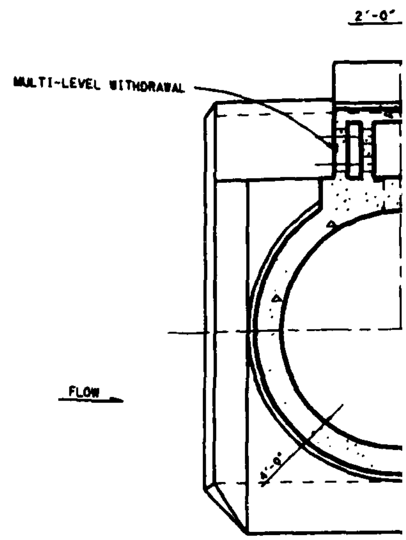
DISTRICT FILE NO. OAK-20-1/4

PLATE 2-4

CORPS OF ENGINEERS 4 3

D

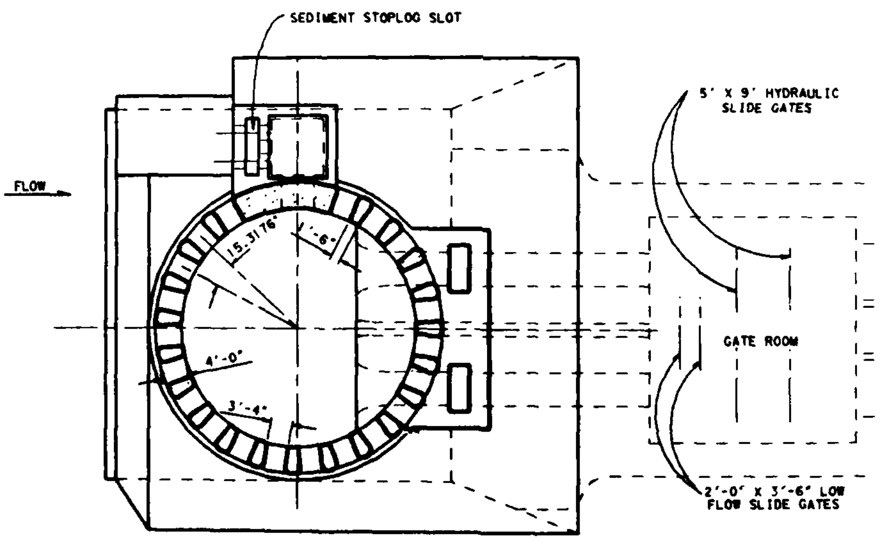


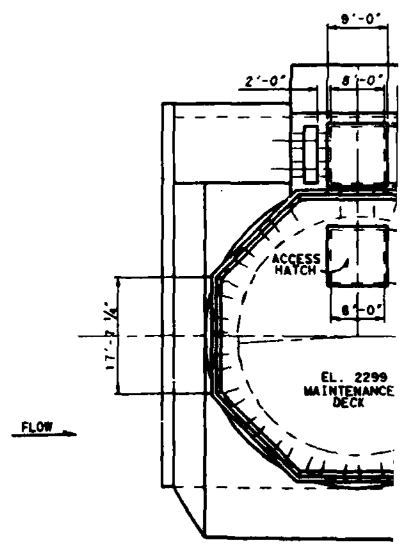


C

SECTION C/1-3
SCALE: 1" = 10'
10' 0 10'

B



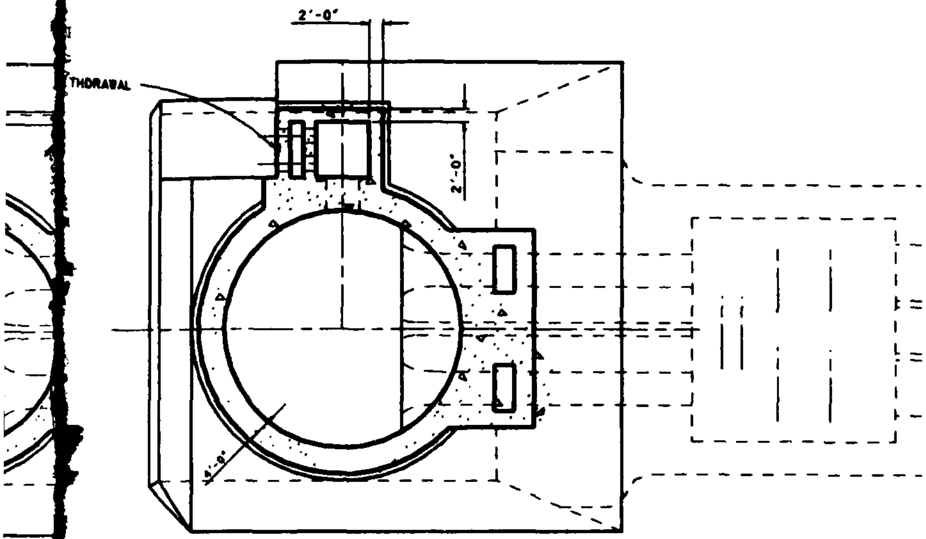


A

SECTION E/1-3
SCALE: 1" = 10'
10' 0 10'

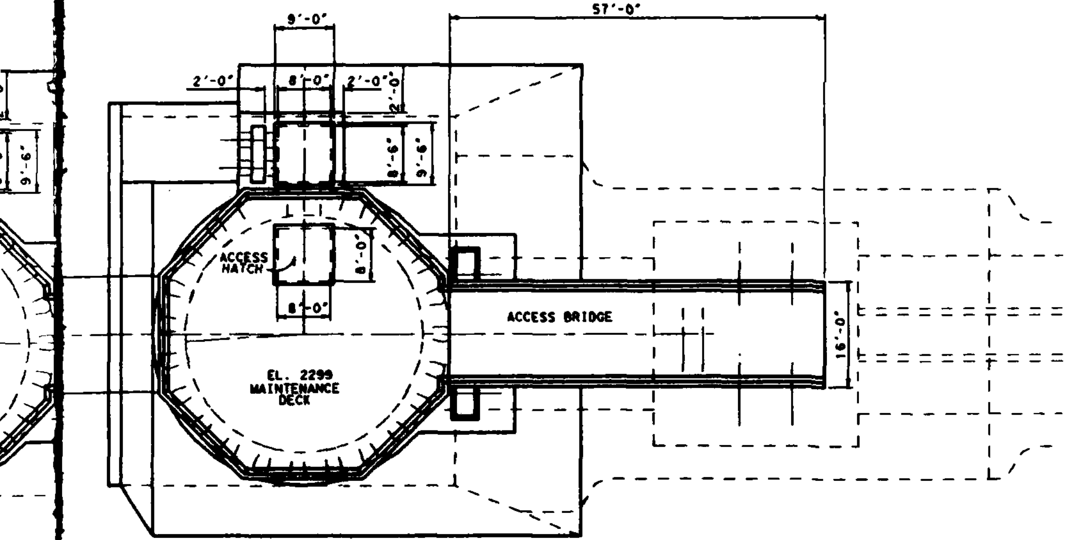
4 3

U.S. ARMY



SECTION D/1-3

SCALE: 1" = 10'



SECTION F/1-3

SCALE: 1" = 10'

REVISIONS

U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS	U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS

DESIGNED BY: D. CHAMBERS

DRAWN BY: S. ERICSON

CHECKED BY: S. KONNO

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
OUTLET WORKS

UPSTREAM CONTROL

INTAKE TOWER
PLAN SECTIONS SHEET 2

SUBMITTED BY: W. E. DAUGHERTY, P.E. CHIEF, DESIGN BRANCH

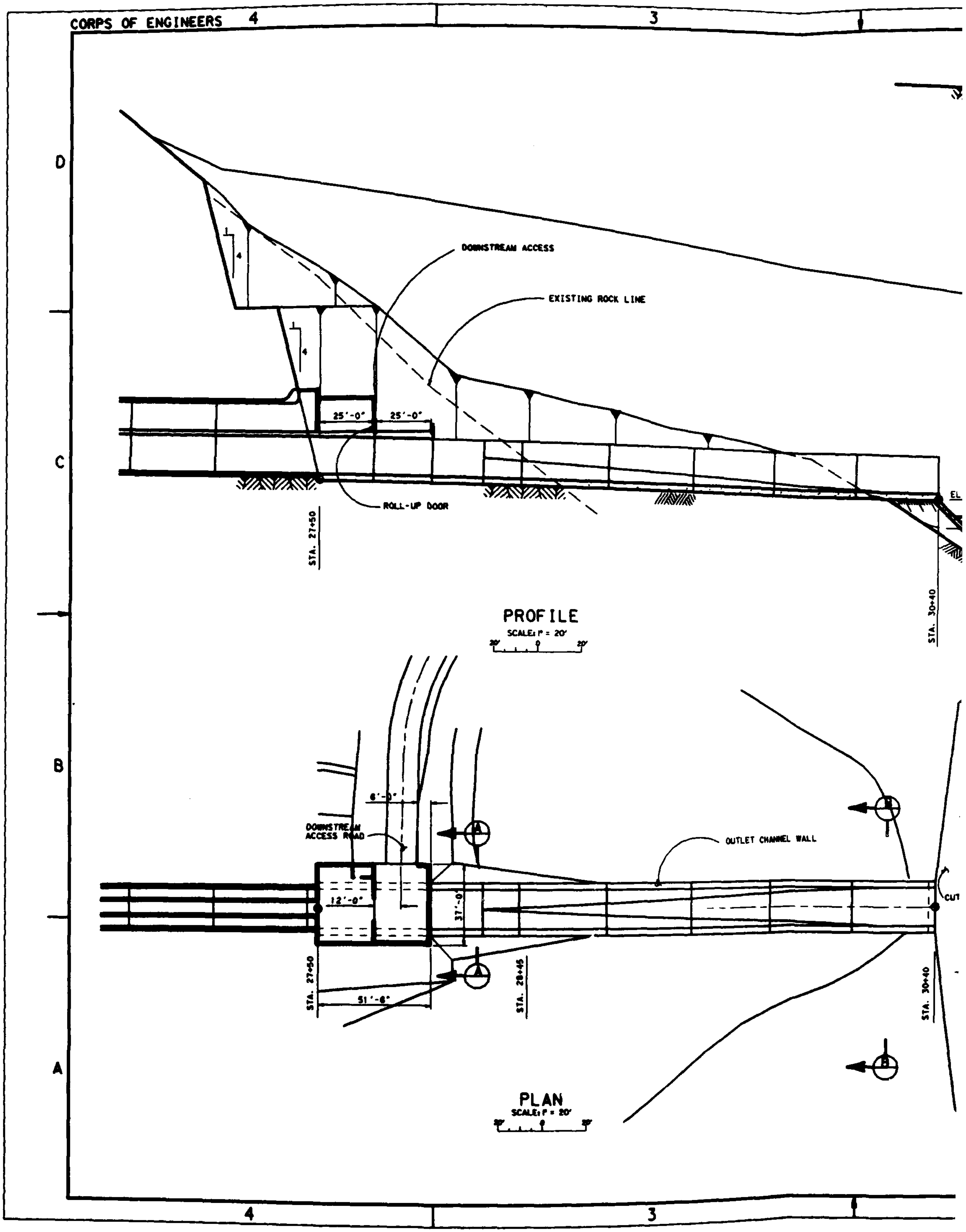
DATE APPROVED:

SPEC. NO. DACW09-___ B-___

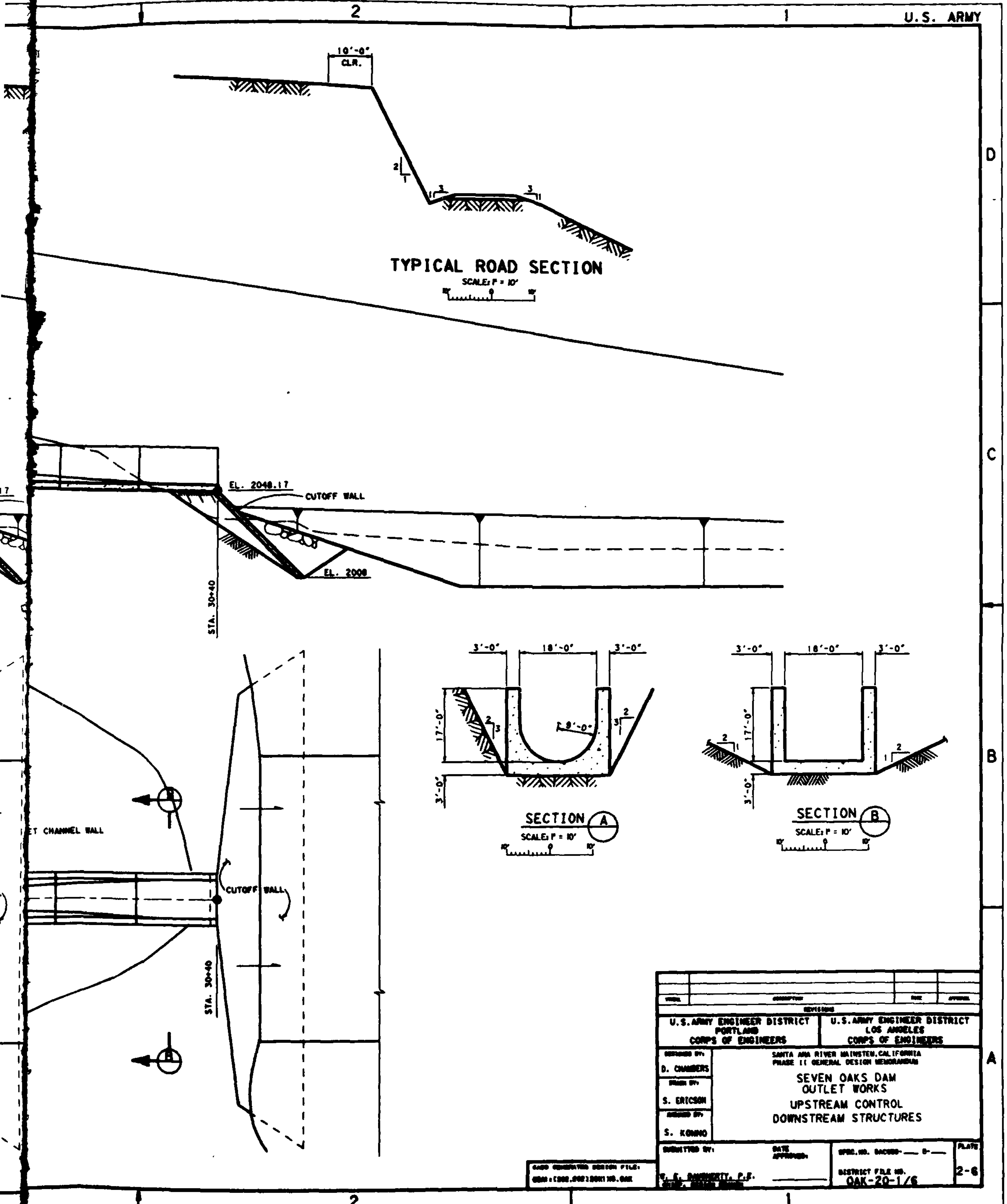
DISTRICT FILE NO. OAK-20-1/5

PLATE 2-5

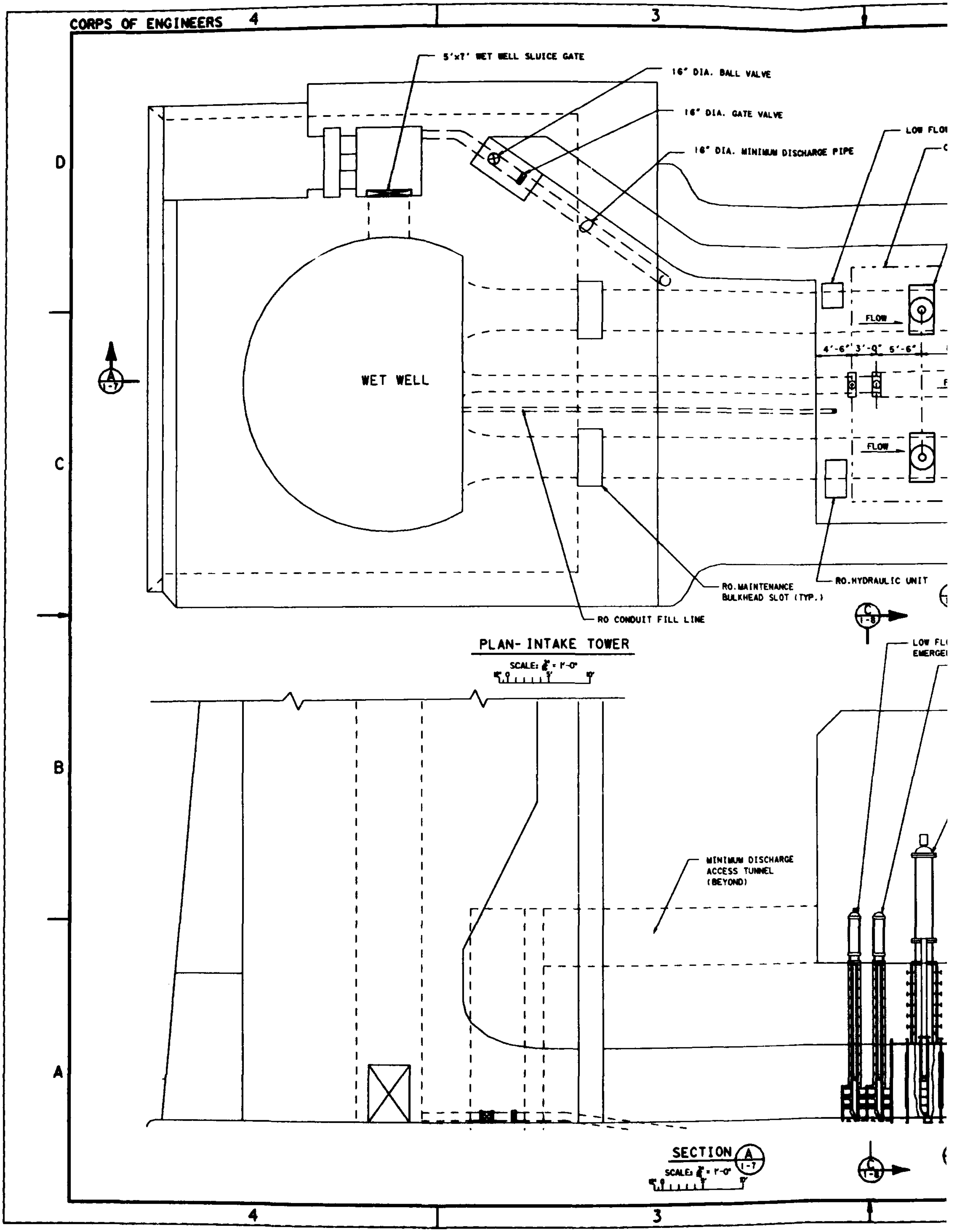
CADD GENERATED DESIGN FILE:



CORPS OF ENGINEERS
DOWNSTREAM ACCESS
EXISTING ROCK LINE
25'-0"
25'-0"
ROLL-UP DOOR
STA. 27+50
STA. 30+40
PROFILE
SCALE: 1" = 20'
DOWNSTREAM ACCESS ROAD
OUTLET CHANNEL WALL
12'-0"
37'-0"
51'-6"
STA. 27+50
STA. 28+45
STA. 30+40
PLAN
SCALE: 1" = 20'

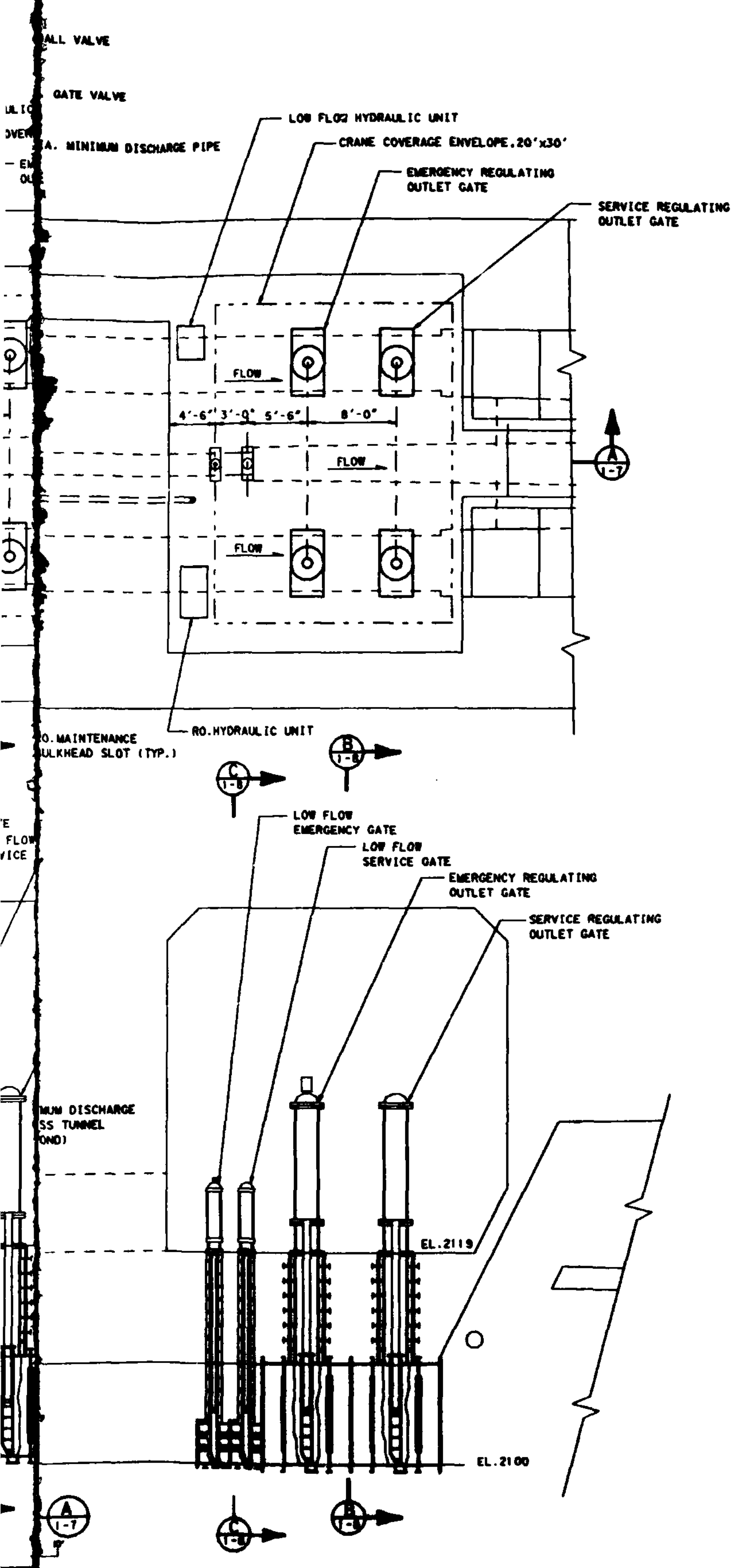


U.S. ARMY
2
1
D
C
B
A
10'-0"
CLR.
TYPICAL ROAD SECTION
SCALE: 1" = 10'
EL. 2048.17
CUTOFF WALL
EL. 2008
STA. 30+40
3'-0"
18'-0"
17'-0"
9'-0"
SECTION A
SECTION B
CHANNEL WALL
CUTOFF WALL
REVISIONS
U.S. ARMY ENGINEER DISTRICT
PORTLAND
CORPS OF ENGINEERS
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM
D. CHAMBERS
S. ERICSON
S. KONNO
SEVEN OAKS DAM
OUTLET WORKS
UPSTREAM CONTROL
DOWNSTREAM STRUCTURES
SUBMITTED BY:
DATE APPROVED:
DISTRICT FILE NO.
OAK-20-1/6
PLATE
2-6



CORPS OF ENGINEERS
4
3
D
C
B
A
5'x7' WET WELL SLUICE GATE
16" DIA. BALL VALVE
16" DIA. GATE VALVE
16" DIA. MINIMUM DISCHARGE PIPE
LOW FLOW
WET WELL
FLOW
FLOW
4'-6" 3'-0" 5'-6"
RO. MAINTENANCE BULKHEAD SLOT (TYP.)
RO. HYDRAULIC UNIT
RO CONDUIT FILL LINE
PLAN- INTAKE TOWER
SCALE: 1/8" = 1'-0"
LOW FL
EMERGE
MINIMUM DISCHARGE ACCESS TUNNEL (BEYOND)
SECTION A 1-7
SCALE: 1/8" = 1'-0"
A 1-7
C 1-8

U.S. ARMY



NOTES:

1. SEE DRAWING OAK-20-1/8 FOR FRONT ELEVATION OF SLIDE GATES AND HYDRAULIC SCHEMATIC DIAGRAM.

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS

U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
OUTLET WORKS
UPSTREAM CONTROL
MECHANICAL EQUIPMENT LAYOUT

DESIGNED BY: R.M.
DRAWN BY: D.B.
CHECKED BY: R.M.
SUBMITTED BY: W. E. DAUGHERTY, P.E.
DATE APPROVED:
SPEC. NO. DACW09-__ B-__
DISTRICT FILE NO. OAK-20-1/7
PLATE 2-7

CADD GENERATED DESIGN FILE:

2 | 1

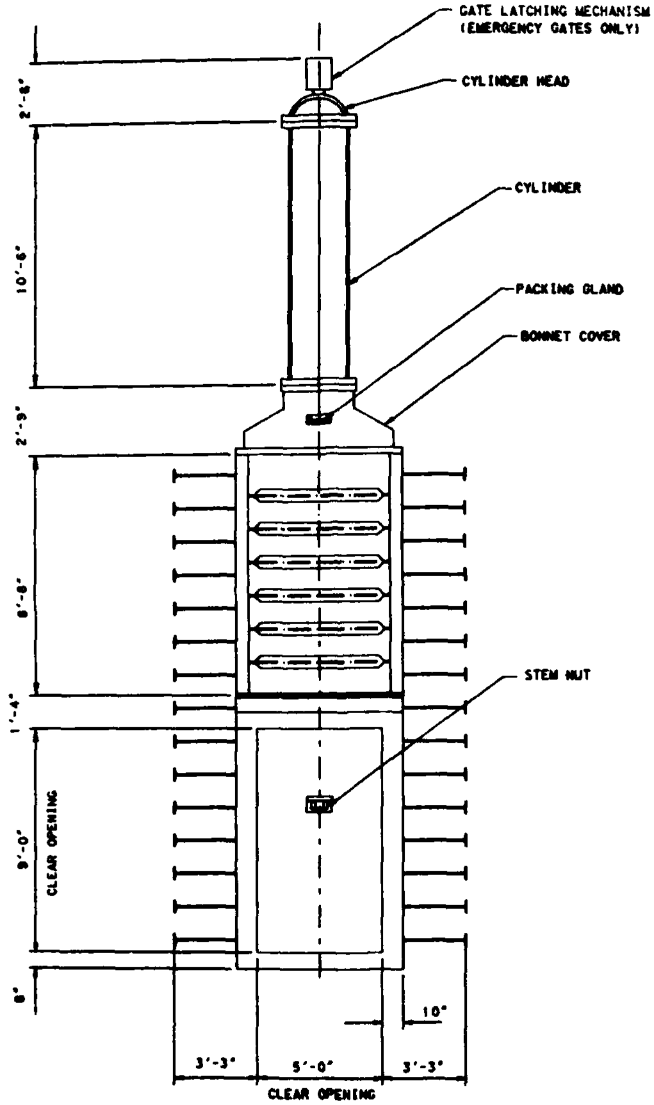
CORPS OF ENGINEERS 4 3

D

C

B

A

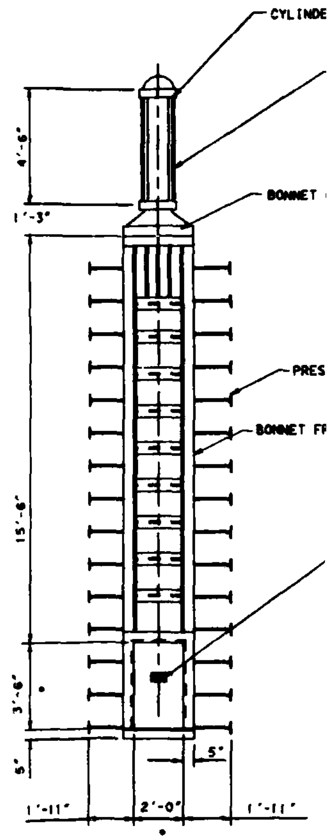


SECTION B 1-7 B 3-8 B 4-8

SCALE: 3/8" = 1'-0"

FRONT ELEVATION

REGULATING OUTLET GATE



SECTION C 1-7 C 3-8 C 4-8

SCALE: 3/8" = 1'-0"

FRONT ELEVATION

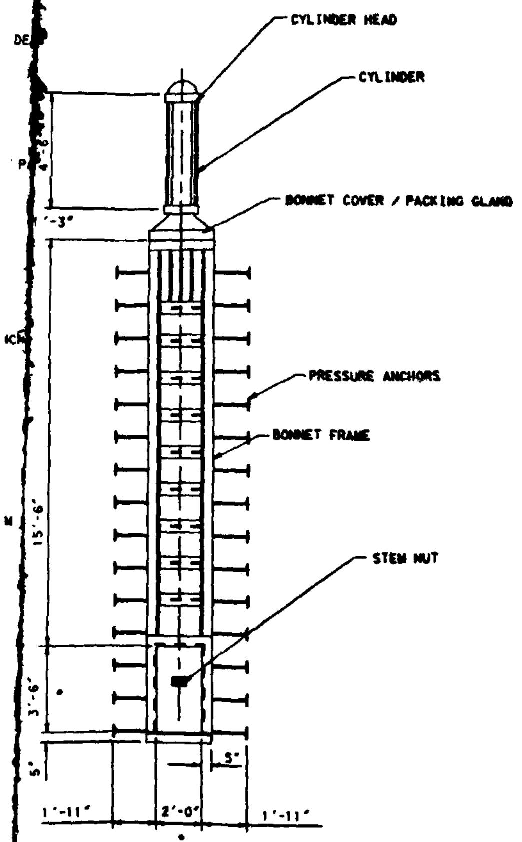
LOW FLOW BYPASS GATE

* CLEAR OPENINGS

4 3

U.S. ARMY

ITEM	NAME	DESCRIPTION
	OIL FILTER, INDICATING TYPE	
	INDUCTION MOTOR, 1800 RPM	40 HP, 460 V, 3 PHASE
	OIL PUMP 1800 RPM	27 GPM, 1750 RPM, 2000 PSI
	PRESSURE RELIEF VALVE	1500-2500 PSI RANGE
	CYLINDER	DOUBLE ACTING
	CHECK VALVE	75 PSI CRACKING PRESSURE
	SHUT OFF VALVE	BALL OR GLOBE TYPE
	4 INCH PRESSURE GAGE	0-3000 PSI
	PRESSURE SWITCH	1500-2500 PSI RANGE
	FILTER AND BREATHER CAP	
	FOUR WAY VALVE	SPR.CT.OPEN CTR. CYL.PORTS BLOCKED
	COUNTERBALANCE VALVE	200-500 PSI RANGE
	AIR BLEED VALVE	
	EMERGENCY GATE LATCH	
	GLOBE VALVE	



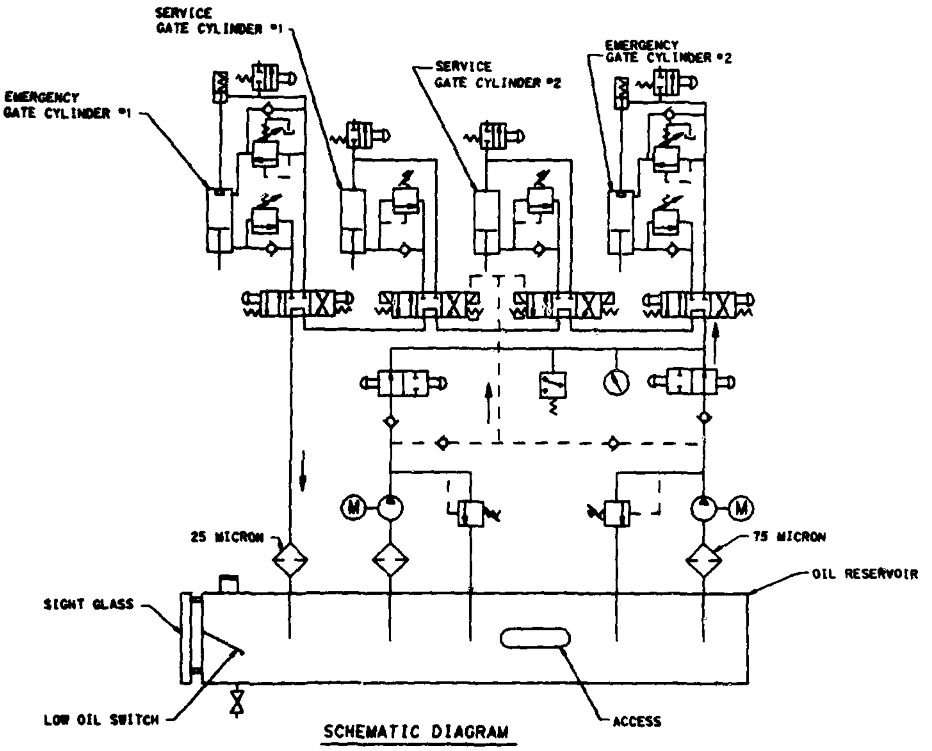
SECTION C 1-7 C 3-8 C 4-8

SCALE: 3/8" = 1'-0"

FRONT ELEVATION

LOW FLOW BYPASS GATE

* CLEAR OPENINGS



SCHEMATIC DIAGRAM

(LOW FLOW GATE SIMILAR)

NO SCALE

THIS PRINT REDUCED TO ONE-HALF SCALE

U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS

U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
OUTLET WORKS

RO GATE AND SCHEMATIC

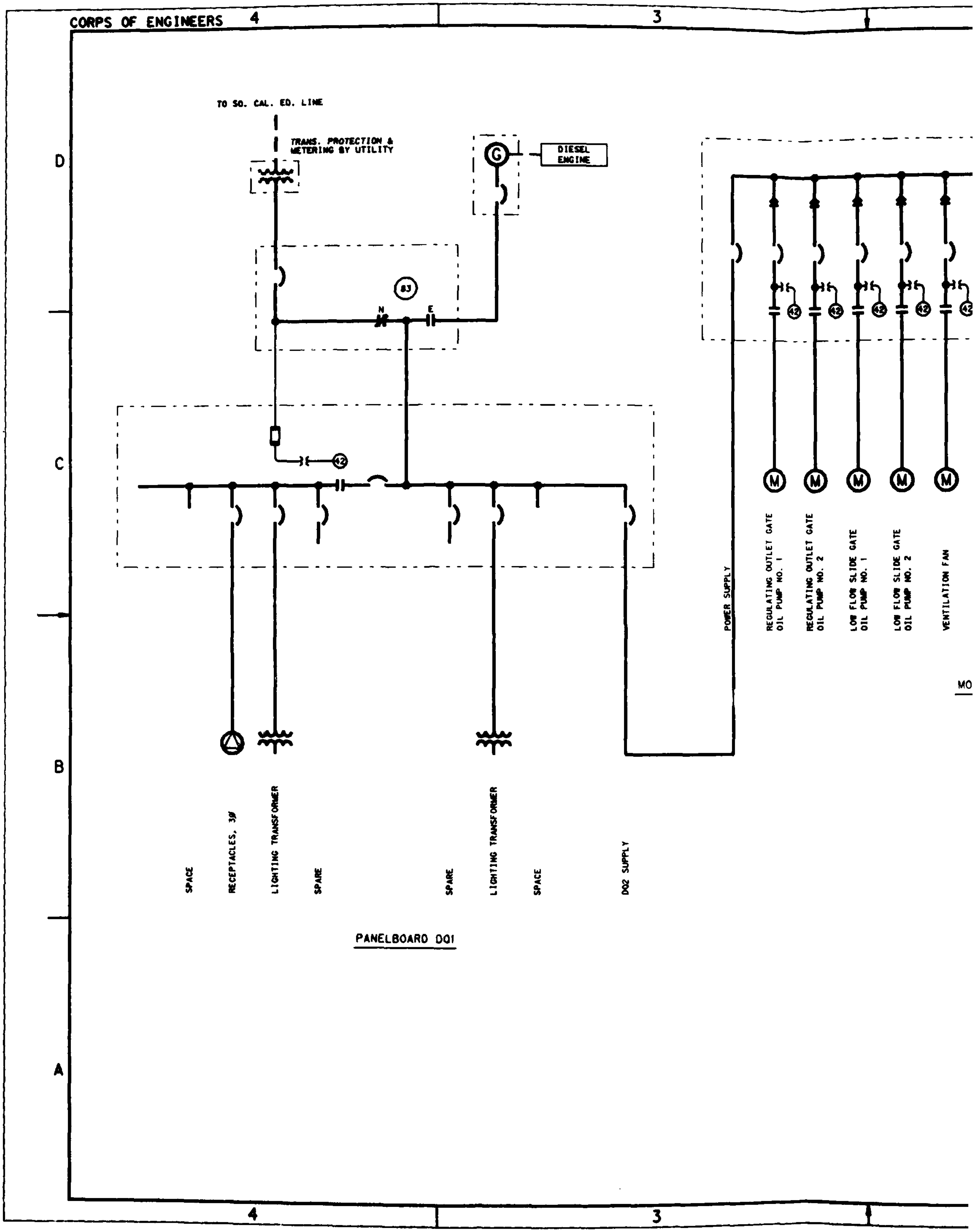
DESIGNED BY: KST

DRAWN BY: DAB

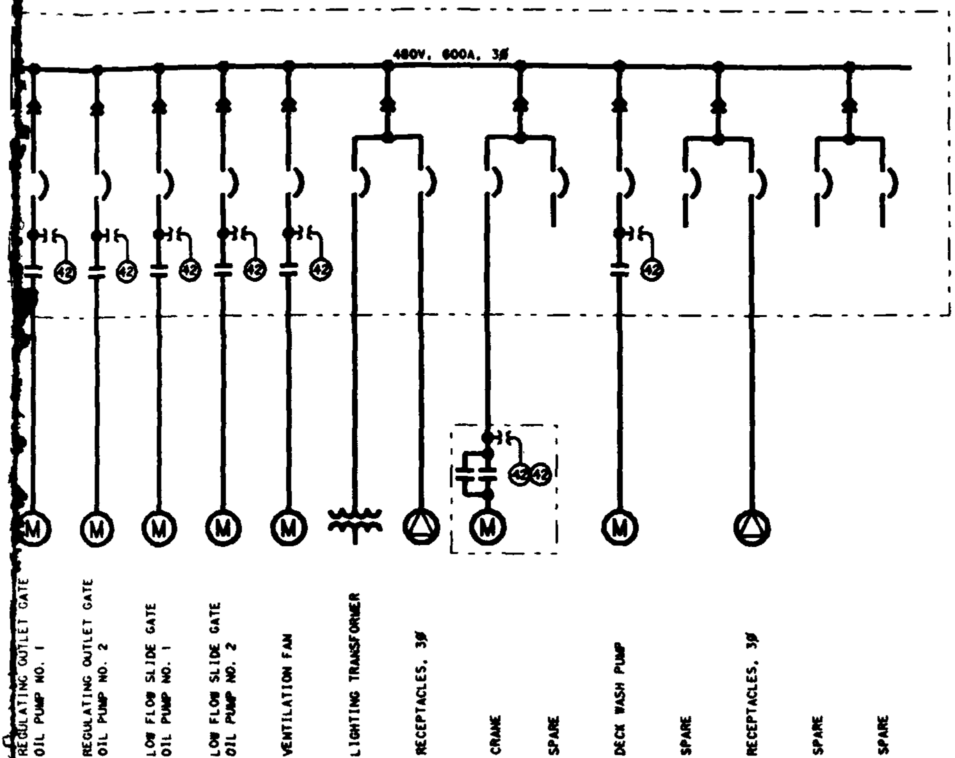
CHECKED BY: RAM

DISTRICT FILE NO. OAK-20-1/8

PLATE 2-8


CORPS OF ENGINEERS
TO SO. CAL. ED. LINE
TRANS. PROTECTION & METERING BY UTILITY
DIESEL ENGINE
POWER SUPPLY
REGULATING OUTLET GATE OIL PUMP NO. 1
REGULATING OUTLET GATE OIL PUMP NO. 2
LOW FLOW SLIDE GATE OIL PUMP NO. 1
LOW FLOW SLIDE GATE OIL PUMP NO. 2
VENTILATION FAN
SPACE
RECEPTACLES, 3Ø
LIGHTING TRANSFORMER
SPARE
SPARE
LIGHTING TRANSFORMER
SPACE
DQ2 SUPPLY
PANELBOARD DQ1

U.S. ARMY



MOTOR CONTROL CENTER DQ2

LEGEND

G	GENERATOR
M	MOTOR
	RECEPTACLE, 3∅, 480V
	CIRCUIT BREAKER OR MOTOR CIRCUIT PROTECTOR
42	MOTOR STARTER OR CONTACTOR
83	AUTO-TRANSFER SWITCH E-EMERGENCY N-NORMAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS

U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
OUTLET WORKS
UPSTREAM CONTROL
ONE-LINE DIAGRAM

DESIGNED BY: RTO
DRAWN BY: JLN
CHECKED BY: KAB

SUBMITTED BY: D. E. DAUGHERTY, P.E.

DATE APPROVED: ______

SPEC. NO. DACW09-___ B-___

DISTRICT FILE NO. OAK-20-1/9

PLATE 2-9

SECTION 3

DOWNSTREAM CONTROL - OUTLET WORKS

3.1 General. This section describes the regulating outlet (RO) features for an outlet works with the control located at the downstream tunnel portal (see General Plan, plate 3-1). The system is comprised of a high level intake tower, a RO/diversion tunnel, a pressurized steel RO conduit, a downstream control and equipment structure located at the tunnel portal, and an outlet channel connecting the control structures to an energy dissipating plunge pool.

3.2 Intake.

a. General. The intake tower is a reinforced concrete structure with a maximum height of 222.5 feet partially embedded into a dioritic rock formation (see plate 3-3). The lower 76 feet of the tower is surrounded by rock on three sides. The structure cantilevers above El. 2,156 as a freestanding tower for 146.5 feet. The tower height was set based on an expected sediment deposition over the project life of 165 feet, or from El. 2,100 to El. 2,265. The sill of the intake tower for floodflows (normal operating maximum of 8,000 cubic feet per second [cfs]) is located at the 2,265 elevation. The tower is designed for operation under submerged conditions. The top 146.5 feet of the tower is essentially a light circular structure with a maintenance deck and access bridge at El. 2,299. Below El. 2,156, the tower is characterized by a more massive rectangular section which houses the multilevel withdrawal wet well, RO wet well, maintenance and emergency gating, and conduit entrances and transitions. Below El. 2,265, on the left side of the tower, is the multilevel withdrawal system. This system is a series of small diameter intakes used to regulate the lower debris pool. Outflow from the withdrawal system discharges into the large 36-foot-diameter tower wet well. The debris pool after 100 years of reservoir sedimentation or a flood event at year zero will see the tower submerged, while a standard project flood (SPF) will submerge the tower by approximately 280 feet.

b. Approach Channel. For the first operating years, the diversion approach channel will become the operating RO intake channel. The channel will be approximately 30 feet wide at the bottom, with alluvial side slopes of 1V on 2H. The elevation of the channel invert at the tower will be 2,097 rising back naturally to the river channel upstream of the dam site at an approximate channel length of 1,040 feet and at a 3 percent slope.

c. Access. Tower access to the maintenance deck is required to perform yearly inspection, maintenance, bulkheading, and debris handling. The tower will be accessible from a road off of the top of the embankment dam from the right abutment across the upstream dam face, to the left abutment where it is cut into rock, to a point immediately downstream of the tower. The final access leg is accomplished by a 54-foot span single-lane bridge to the El. 2,299 maintenance deck. The road is approximately 4,800 feet long, sloped at an average of 6 percent, single lane (with turnouts), and paved. The road will be designed for project crane and dump truck operations. Longer bridges were investigated to minimize rock excavations, but preliminary analysis found associated pier heights to be prohibitive in this seismic environment. Other road alignments on the left abutment were found to have excessive length and rock excavation. There was also the Edison penstock crossing to consider.

d. High Level Intake. The primary intake is designed to pass the regulated higher flows in accordance with the operating criteria as depicted in section 3. Upwards of 8,000 cfs can be passed as a normal design flow through the 116 3-foot 4-inch-square openings covering an area from El. 2,265 to El. 2,292.5. The openings are separated on all sides by 1-foot 6-inch beams (trash struts). Openings extend 315 degrees around the circumference of the intake tower. Eight hundred square feet are required for entrance velocity conditions; an additional 489 square feet have been provided as a safety factor against plugging. Flow past the trash struts enters a 36-foot-diameter wet well. In the ceiling of the intake is an 8-foot-square access hatch, provided for inspection, cleanout, and any maintenance required.

e. Wet Well. Within the tower is a 36-foot-diameter circular wet well. It extends for the full tower height, some 195 feet. The walls of the tower are constructed on top of a 20-foot-thick massive rectangular base slab. The walls are mass concrete from the base at El. 2,100 up to El. 2,156. In plan view, the tower below El. 2,156 is rectangular on the outside. Above El. 2,156, the tower is circular with a 5-foot-thick wall reducing to 4 feet for the top 84.5 feet. Near the bottom of the tower the downstream face of the wet well flattens to form a vertical plane where the square bell-mouth entrance to the outlet conduit is found (see plate 3-3). Within the upstream face of the tower at El. 2,100, is a 5-foot-diameter conduit which will be used to facilitate maintenance at the intake structure.

f. Regulating Outlet. The main RO will handle the larger flows, primarily flows above 300 cfs under normal operating conditions. The entrance to the RO conduit is found at the bottom of the large wet well. The opening is an 11-foot square bell mouth opening which transitions (20-foot transition) to an 11-foot diameter steel pressure conduit. The RO conduit will have a 4-foot by 15-foot bulkhead slot located 12 feet downstream of the bell-mouth entrance. The maintenance bulkhead would be stored in the slot. The bulkhead slot is required to extend upward to an elevation above the El. 2,265 expected sediment level. The bulkhead will be a slide gate capable of being lowered by crane under submerged conditions of a reservoir elevation less than the maintenance deck at El. 2,299.

g. Low Flow Bypass. The entrance to the low flow bypass is located at the bottom of the large wet well, to the right of the main RO conduit (see plate 3-4). The entrance is a 60-inch square bell-mouth which transitions to a 42-inch steel conduit. The opening will be either gated for maintenance or have provisions for bulkheading and will have a trashrack with a 1-foot bar spacing. This line in conjunction with the minimum discharge line will be used for low flow diversion during the summer tunnel installation of the main RO conduit.

h. Multilevel Withdrawal System. The multilevel withdrawal system (MWS) consists of two columns of 2-foot 3-inch-diameter ports spaced at

10 vertical feet for the full height of the sediment range (El. 2,100 to El. 2,265). The ports are located on the left side of the tower and have horizontal column spacing of 4 feet 6 inches on centerline. This system regulates the reservoir storage area below the intake tower sill at El. 2,265. The MWS ports are covered with trashrack grating and discharge into an 8-foot 6-inch by 8-foot wet well. An access hatch for maintenance will be located at the top of the wet well. The wet well discharges into the larger 36-foot wet well through a 5- by 7-foot conduit at El. 2,100. A large manually operated wet well sluice gate will be located at the conduit entrance with an operator located above El. 2,265. This gate will not be used as a throttling gate, because it will be used either fully opened or fully closed. A stoplog slot is located between the trashracks and wet well. Prior to flood season, sediment stoplogs will be installed. The concrete stoplogs will be placed in advance of the rising sediment (approximately 20 to 30 feet above the sediment level to account for the predicted rise in sediment depth from an SPF event). With the stoplogs utilized, sediment passage through the project will be minimized.

i. Minimum Discharge Line. Flows between 10 and 100 cfs will be passed through a minimum discharge line (39-inch pipeline) originating at the bottom of the MWS wet well (see plate 3-4). These flows cannot be accommodated by the hydraulic slide gates between pool Els. 2,100 and 2,350, see figures 6-1 and 6-2. An upstream sluice gate or bulkheading will be provided for maintenance. The minimum discharge line transitions to a 14-inch line within the downstream control structure. Control of flow is achieved using a 14-inch valve with an emergency ball valve for emergency backup.

3.3 Regulating Outlet/Diversion Tunnel.

a. General. The outlet tunnel is 1,623 feet in length and will be constructed as a horseshoe tunnel section utilizing conventional drill and blast techniques (see plate 3-2). The dimensions of the tunnel will be 18 feet wide and 18 feet high within the inside face of the concrete liner. The liner is required for diversion efficiency (smooth wall roughness coefficient). Once the embankment dam nears the standard project flood height, an RO steel conduit will be installed in the

diversion tunnel. This conduit will be 11 feet in diameter and is designed similar to a pressure penstock. Conduit installation will take place during the low flow months when the handling of water will be through the 3-foot 6-inch and 3-foot 3-inch steel pipes located in the floor of the tunnel. The tunnel has a constant slope of 0.026.

b. Upstream Transition, Zone A. The first 150 feet of the tunnel contains the conduit transition from a square steel section to an 11-foot-diameter circular section. This portion of the tunnel will have the upstream concrete plug. The area between the diversion liner and the conduit will be plugged with reinforced concrete. The rock mass at inlet will be grouted and drained. The remaining portion of zone A will have an 18-inch reinforced concrete liner (reference section B, plate 3-2).

c. Zone B, Station 12+90 to Station 19+00. Roughly 600 feet of the tunnel, upstream of the dam centerline, will have a lightly reinforced 12-inch concrete liner. Contact grouting and a grout ring at the dam centerline is proposed. Minimum provisions for drainage will be provided and a continuous gravel floor drain is planned for external pressure relief. Minimal external hydrostatic loading is expected in this zone (reference section C, plate 3-2).

d. Zone C, Station 19+00 to Station 27+57. The remaining 857 feet of tunnel has an unreinforced concrete liner. Contact grouting will be done between rock and concrete. At the downstream portal the tunnel liner will be reinforced for the expected portal rock loads. The downstream transition from 11 feet circular to 11 feet square takes place just before exiting the tunnel. No external hydrostatic loads are expected in this zone. The tunnel space between liner and conduit will be provided with a man adit, air passage, and nominal floor drainage.

3.4 Downstream Control and Outlet Structures.

a. Control Structures.

(1) General. The downstream control structures are comprised of three monoliths. The first houses conduit transitions, tunnel access,

and acts as the gate structure access deck. The second monolith, the gate structure, is 38-foot square and houses all control gating and related project equipment (diesel generator, reservoir readouts, control annunciation, HVAC, etc.), restroom, office, and storage (see plate 3-6). A large roll-up door is provided to transport project equipment. The larger gate components can be moved through access hatches located in the roof. The third monolith channelizes the flow downstream, serves as a crane deck, and has a large air grating to provide air for all hydraulic gate aeration.

(2) Hydraulic Slide Gates. The two 5- by 9-foot operating slide gates are located approximately 90 feet downstream of the tunnel portal. Emergency slide gates are located 8 feet upstream of the operation gates. The smaller 2-foot by 3 1/2-foot gates are located as shown on plate 3-6. The 2-foot by 3 1/2-foot low flow gates are required in conjunction with the minimum discharge line. The low flow gates discharge lower range flows up to 500 cfs. The minimum discharge valve releases flows of between 10 and 100 cfs. Immediately downstream of the operating gate, air is introduced by use of aeration offsets located about the perimeter of the conduit.

b. Outlet Channel. Exiting from the control structure are four distinct channels. The two larger channels provide passage for the flows being released from the two 5-foot by 9-foot hydraulic slide gates. The channel is broken into four monoliths. Each monolith section is approximately 45 feet long, has a channel width of 11 feet, and has walls 16 feet in height. The smaller channels are 3 feet wide and are used for low bypass and minimum discharge releases (see plate 3-5). The smaller channels will also be used for the summer diversion flows during RO conduit installation.

c. Flip Bucket. At the end of the outlet channel, a flip bucket will be provided to discharge the outflows from the two main regulating outlets. The flip bucket will be approximately 70 feet long and 20 feet high at the lip from the channel invert, with a radius of 120 feet. (See Hydraulics Section of GDM for further details.) Provisions will be made for the flip bucket to drain low flow discharges through the RO outlet channels. Further refinements to the flip bucket trajectory angle and

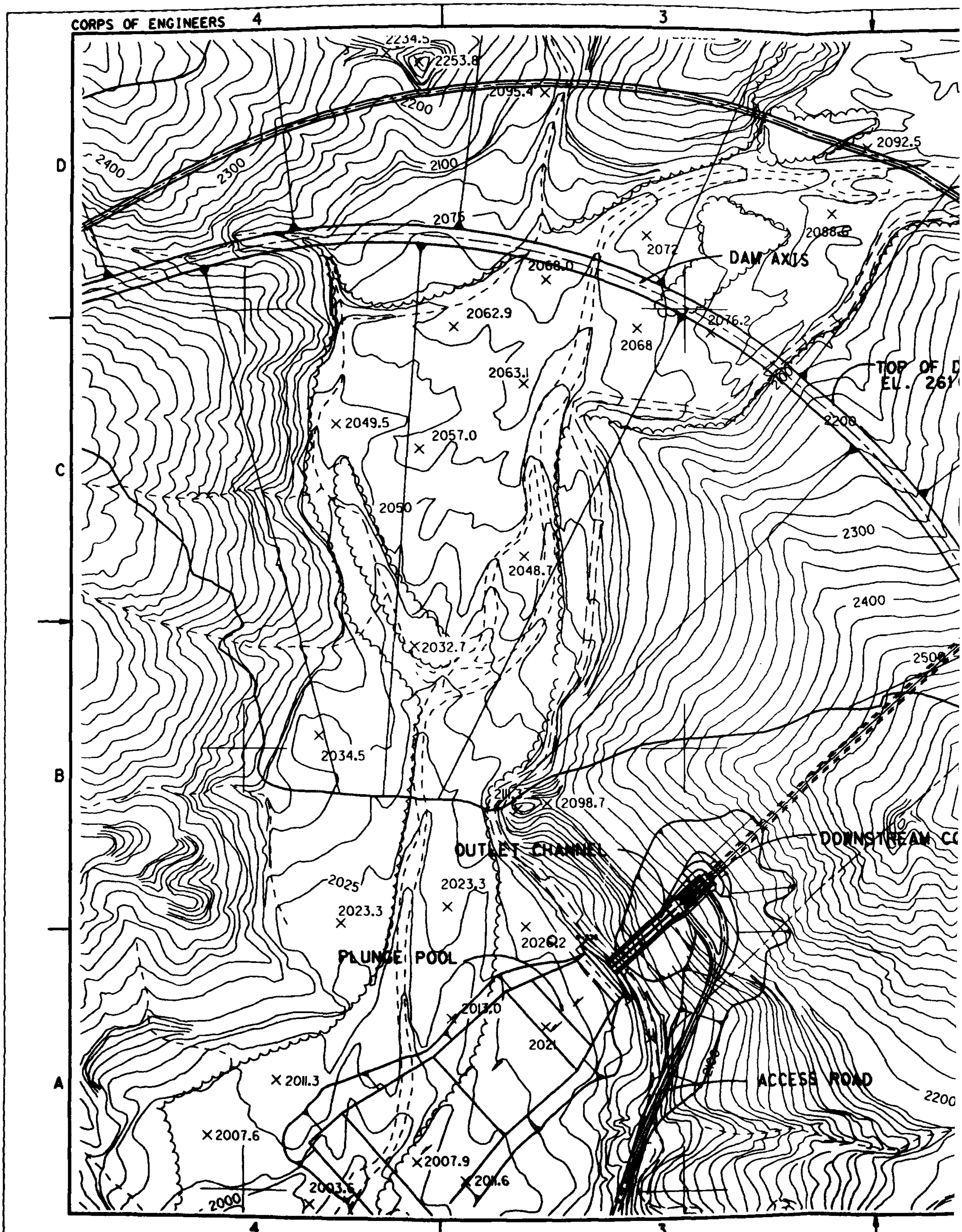
alignment will be investigated in the FDM level to minimize the outlet channel length, plunge pool configuration, and impacts to the downstream access road to the outlet works. The flip bucket will have drains to resist ponding water.

d. Cutoff Wall. The outlet channel terminates approximately 329 feet downstream of the operating slide gate. At this point a cutoff wall/splash slab has been placed on the upstream plunge pool slope. This slope is partially excavated and partially builtup with processed and compacted backfill. The cutoff wall protects against upstream undercutting of the outlet channel structure. To resist uplift, the wall most likely will need to be anchored to the backfill with earth anchors extending through the concrete using pressure grout anchor systems.

e. Plunge Pool. A scour hole will be pre-excavated to two-thirds the depth of the hole that theoretically would form from a constant 8,000 cfs release. This depth will provide initial energy dissipation to protect structures in the downstream channel until the scour hole reaches its equilibrium depth. The estimated final scour hole is a sufficient distance downstream that it will not pose any danger to the dam embankment. It is approximately 740 feet long and varies from 120 feet to 290 feet wide at the bottom with side slopes of 1V on 3H. Larger rock will be left in the basin to accelerate the armoring process.

f. Downstream Access Road. The downstream access road will be a single lane, paved road traversing the left abutment as shown (see general plan).

3.5 Instrumentation. The tower and downstream structures will be typically tied into the project survey monitoring program with key points identified by embedded monuments in the concrete. Tiltmeters and extensometers will be utilized. A seismic accelerograph will be located in the gate room. Hydraulic instrumentation about the slide gates and immediately downstream will be utilized to monitor pressure and flow conditions. A specific plan and types of instrumentation should be developed at the FDM level.



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DAM AXIS
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2062.9
2076.2
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TOP OF D
EL. 261
2063.1
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2057.0
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DOWNSTREAM CO
OUTLET CHANNEL
2025
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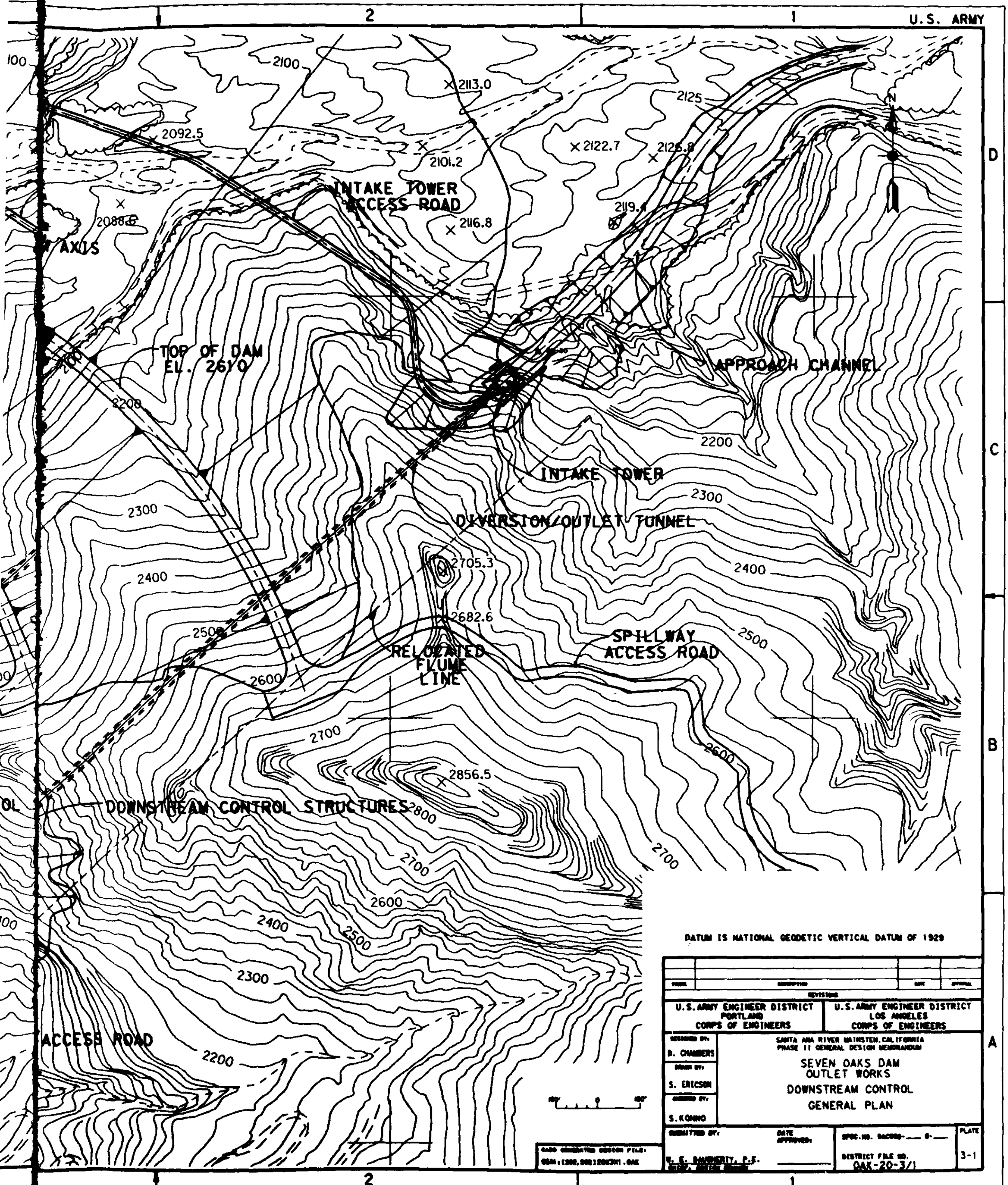

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2119.4
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ACCESS ROAD
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APPROACH CHANNEL
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INTAKE TOWER
2300
DIVERSION/OUTLET TUNNEL
2400
2705.3
2682.6
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ACCESS ROAD
RELOCATED
FLUME
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2700
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DOWNSTREAM CONTROL STRUCTURES
ACCESS ROAD
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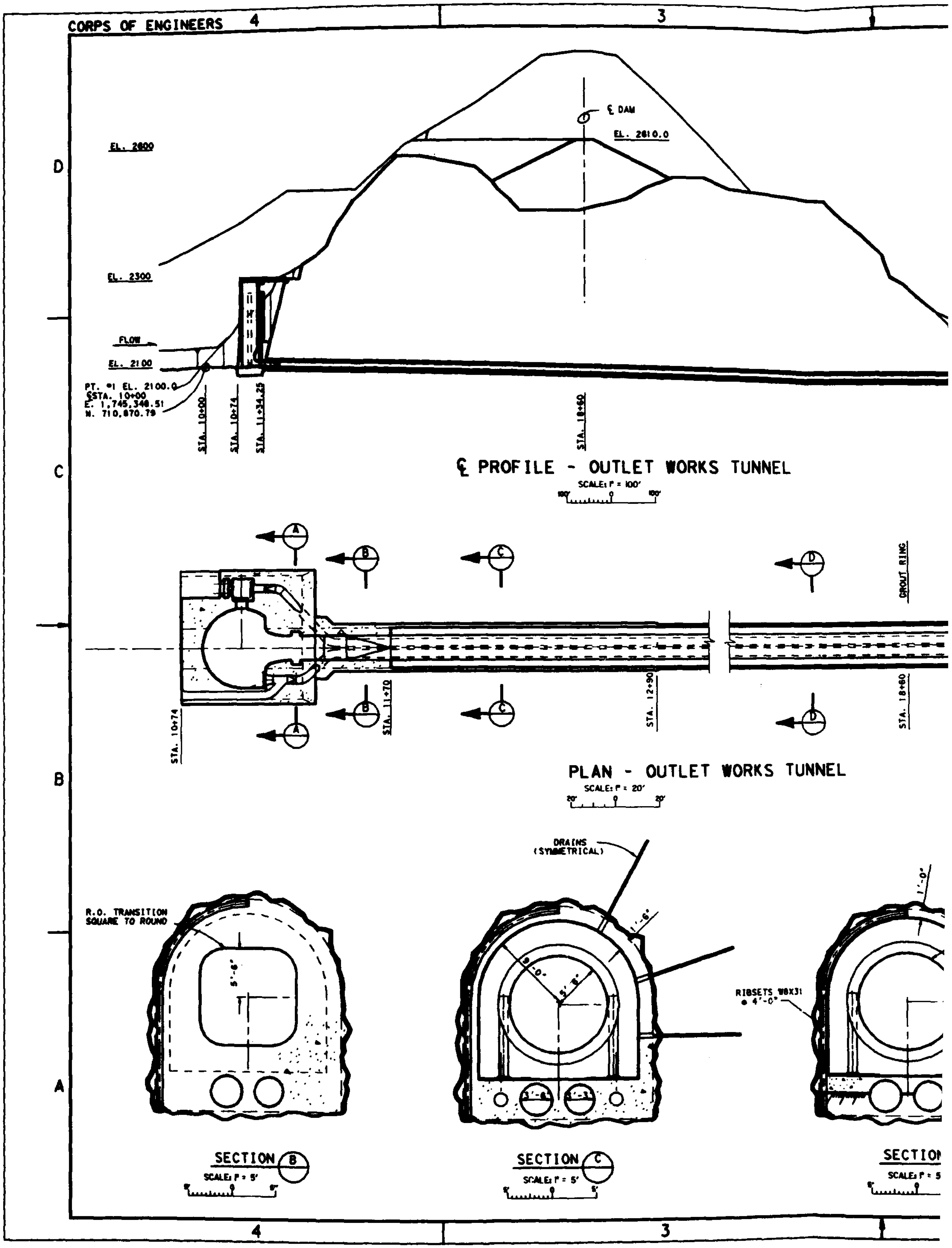
DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929
REVISIONS
U.S. ARMY ENGINEER DISTRICT
PORTLAND
CORPS OF ENGINEERS
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
D. CHAMBERS
S. ERICSON
S. KONNO
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
OUTLET WORKS
DOWNSTREAM CONTROL
GENERAL PLAN
SUBMITTED BY:
DATE APPROVED:
DISTRICT FILE NO.
OAK-20-3/1
PLATE
3-1



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EL. 2600
EL. 2300
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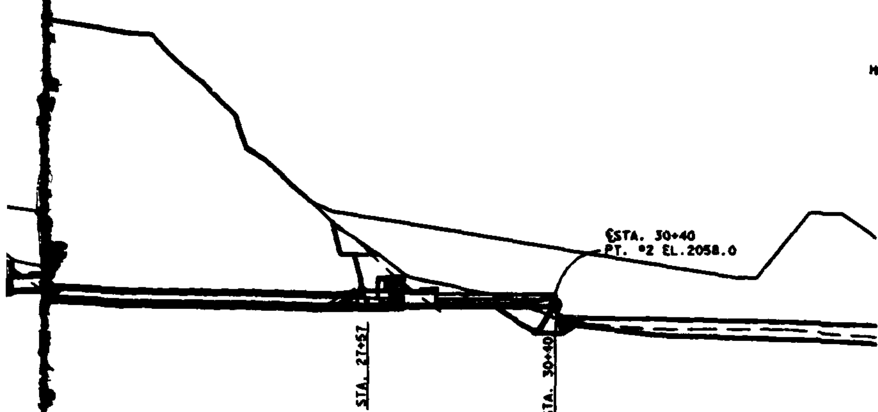
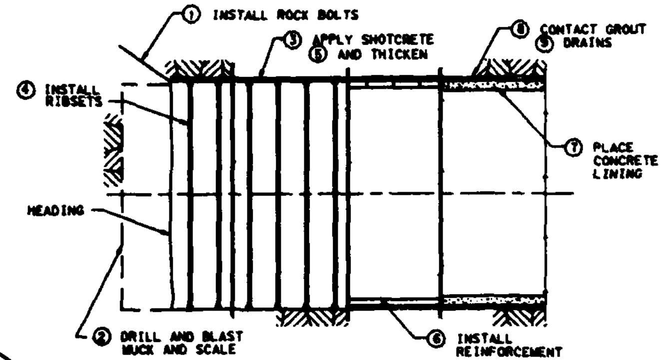
PT. #1 EL. 2100.0
℄STA. 10+00
E. 1,745,348.51
N. 710,870.79
STA. 10+00
STA. 10+74
STA. 11+34.25
STA. 18+60
℄ PROFILE - OUTLET WORKS TUNNEL
SCALE: 1" = 100'
GROUT RING
STA. 10+74
STA. 11+70
STA. 12+90
STA. 18+60
PLAN - OUTLET WORKS TUNNEL
SCALE: 1" = 20'
DRAINS (SYMMETRICAL)
R.O. TRANSITION SQUARE TO ROUND
RIBSETS W8X31 @ 4'-0"
5'-6"
9'-0"
5'-8"
1'-6"
1'-0"
SECTION B
SCALE: 1" = 5'
SECTION C
SCALE: 1" = 5'
SECTION
SCALE: 1" = 5

U.S. ARMY





TUNNELING SEQUENCE

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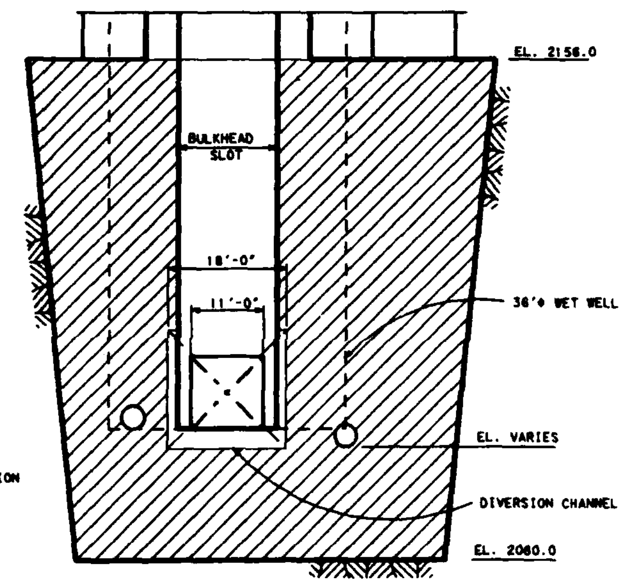

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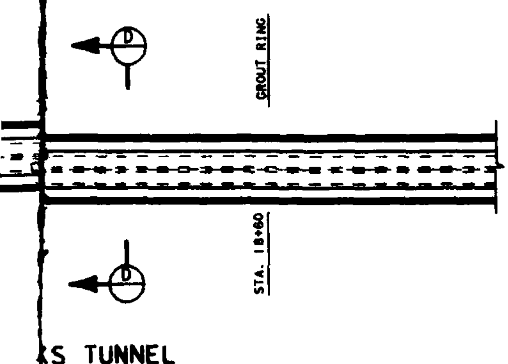
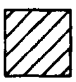
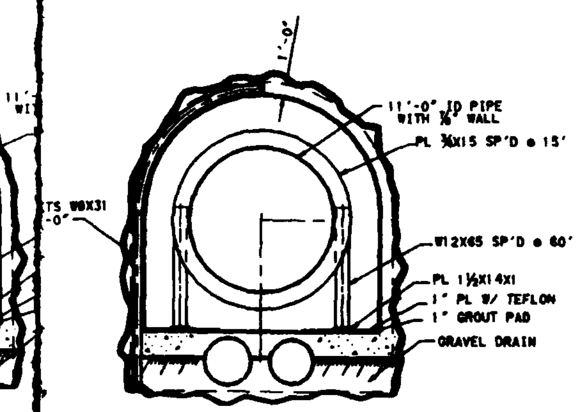




TUNNEL CONTRACT - FIRST STAGE CONSTRUCTION



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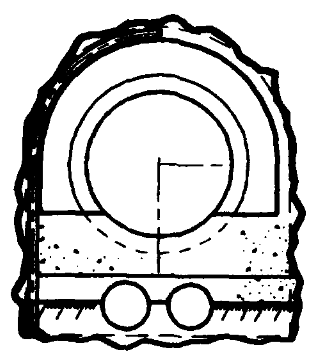

ALTERNATIVE CONDUIT SUPPORT

MARK	DESCRIPTION	DATE	APPROVAL
REVISIONS			

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DESIGNED BY: D. CHAMBERS	SANTA ANA RIVER MAINSTEM, CALIFORNIA PHASE II GENERAL DESIGN MEMORANDUM
DRAWN BY: S. ERICSON	SEVEN OAKS DAM OUTLET WORKS DOWNSTREAM CONTROL
CHECKED BY: K. SWANSON	DIVERSION/OUTLET TUNNEL PLAN, PROFILE & SECTIONS
SUBMITTED BY: W. E. DAUGHERTY, P.E.	DATE APPROVED:
SPEC. NO. DACW09- -B-	PLATE 3-2
DISTRICT FILE NO. OAK-20-3/2	

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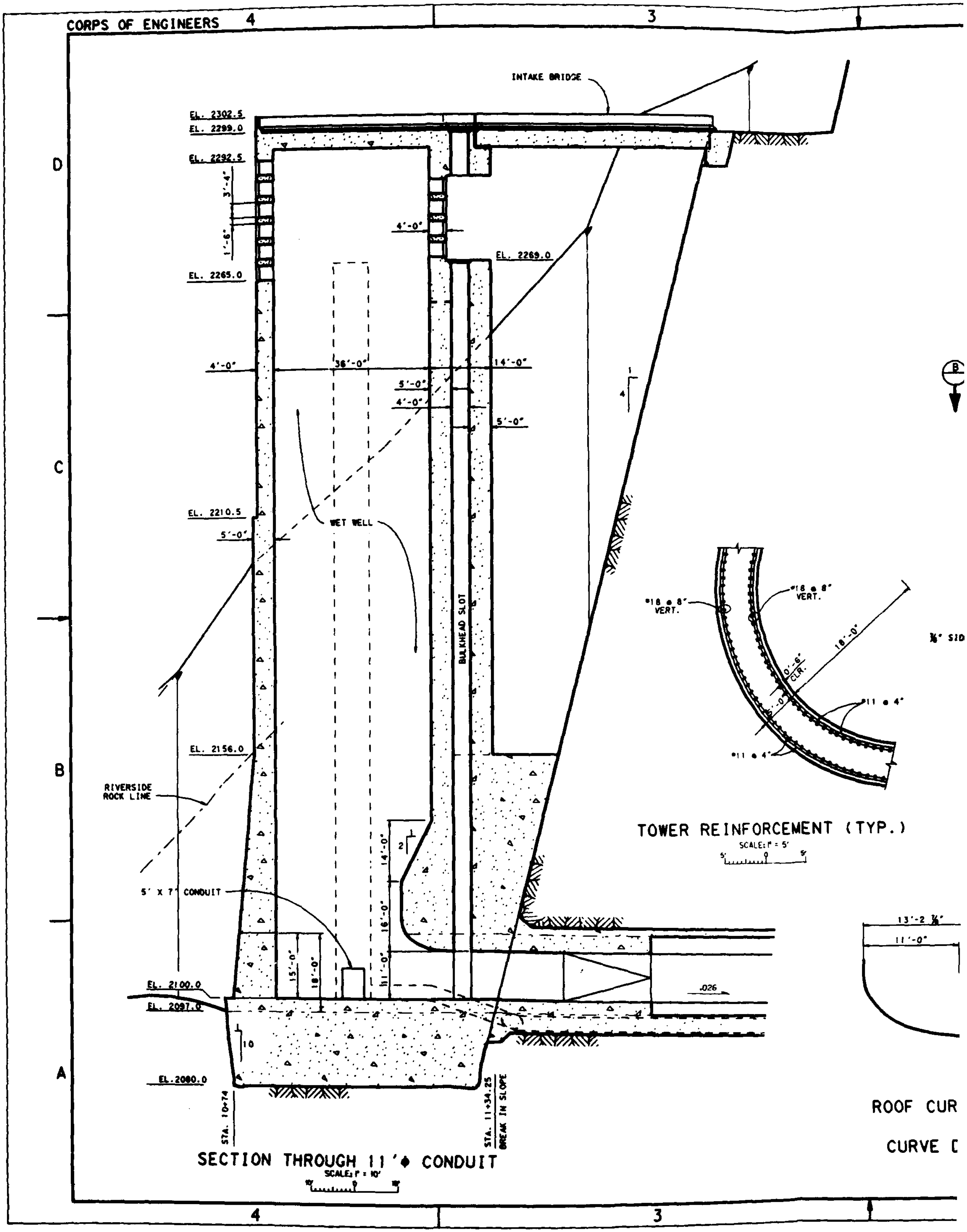
CORPS OF ENGINEERS
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C
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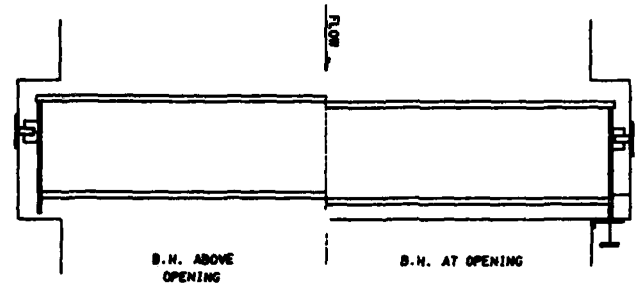
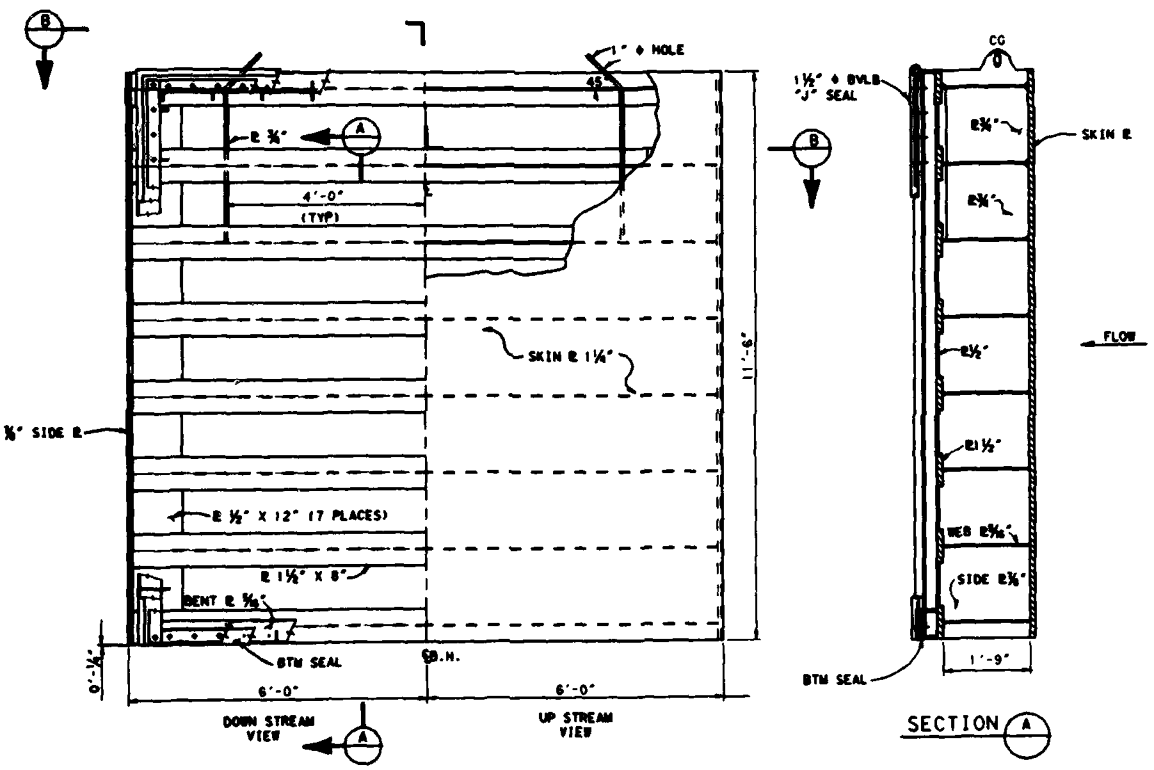
5'-0"
EL. 2210.5
5'-0"
WET WELL
BULKHEAD SLOT
EL. 2156.0
RIVERSIDE ROCK LINE
5' X 7' CONDUIT
14'-0"
16'-0"
15'-0"
18'-0"
11'-0"
EL. 2100.0
EL. 2097.0
.026
10
EL. 2080.0
STA. 10+74
STA. 11+34.25 BREAK IN SLOPE
SECTION THROUGH 11'Φ CONDUIT
SCALE: 1" = 10'
#18 @ 8" VERT.
#18 @ 8" VERT.
18'-0"
0'-6" CLR.
#11 @ 4"
#11 @ 4"
TOWER REINFORCEMENT (TYP.)
SCALE: 1" = 5'
B
13'-2 ¾"
11'-0"
ROOF CUR
CURVE D



SECTION (B)

PLAN - R.O. BULKHEAD



SECTION (A)

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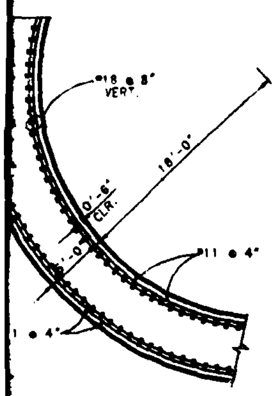

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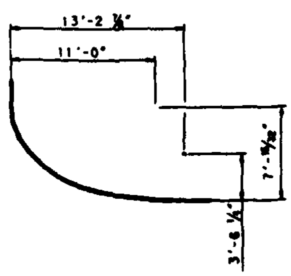
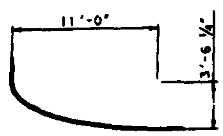


ROOF CURVE



SIDE CURVE

CURVE DATA - 11' X 11' CONDUIT
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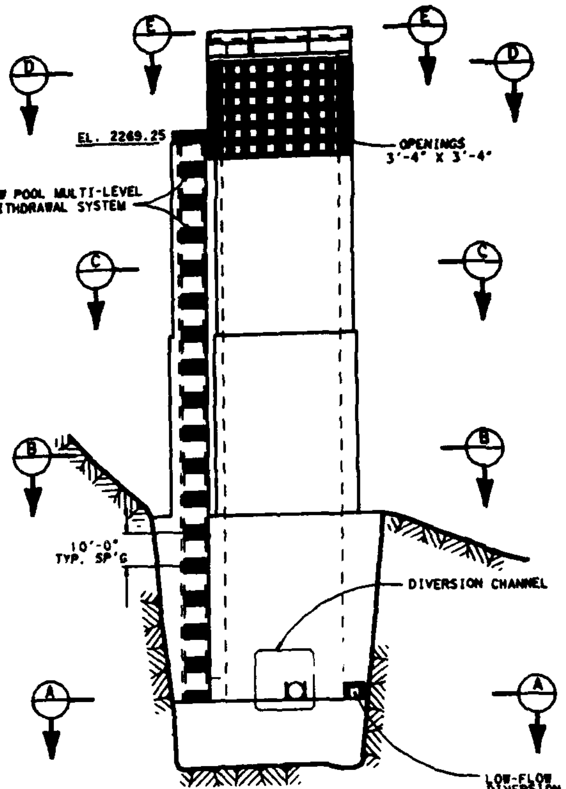
U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS	U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS
DESIGNED BY: D. CHAMBERS DRAWN BY: S. ERICSON CHECKED BY: S. KONNO	SANTA ANA RIVER MAINSTEM, CALIFORNIA PHASE II GENERAL DESIGN MEMORANDUM SEVEN OAKS DAM OUTLET WORKS DOWNSTREAM CONTROL INTAKE TOWER SECTIONS & MISC. DETAILS
SUBMITTED BY: W. E. DAUGHERTY, P.E. CHIEF, DESIGN BRANCH DATE APPROVED:	SPEC. NO. DACW09- B- DISTRICT FILE NO. OAK-20-3/3 PLATE 3-3

CORPS OF ENGINEERS 4 3

D



INTAKE TOWER ELEVATION

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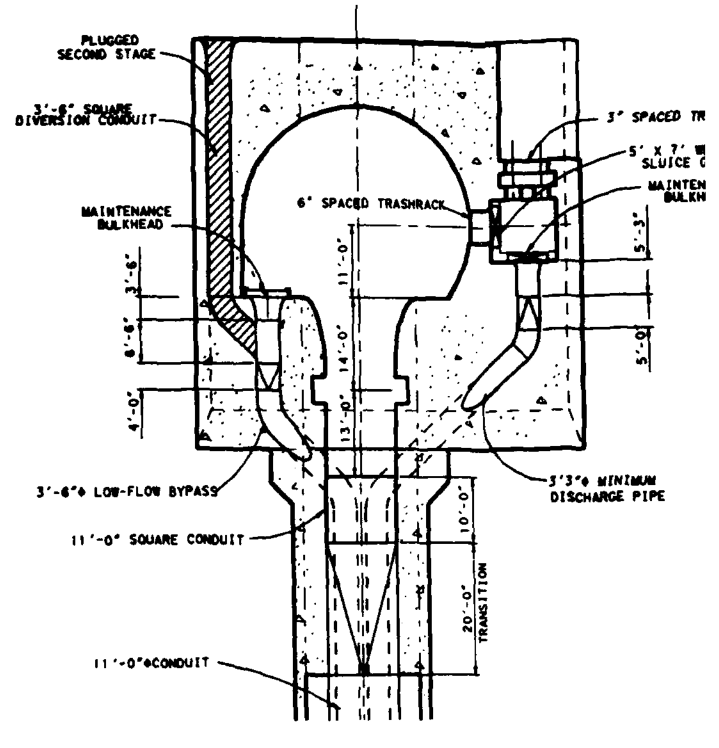

EL. 2103.0

SECTION A

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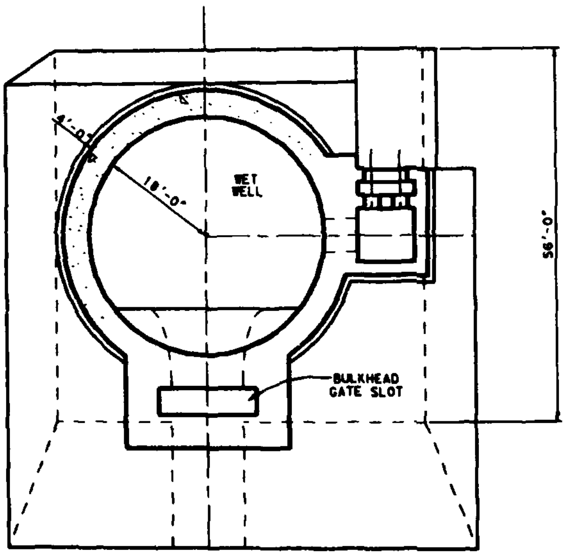
B



EL. 2230.0

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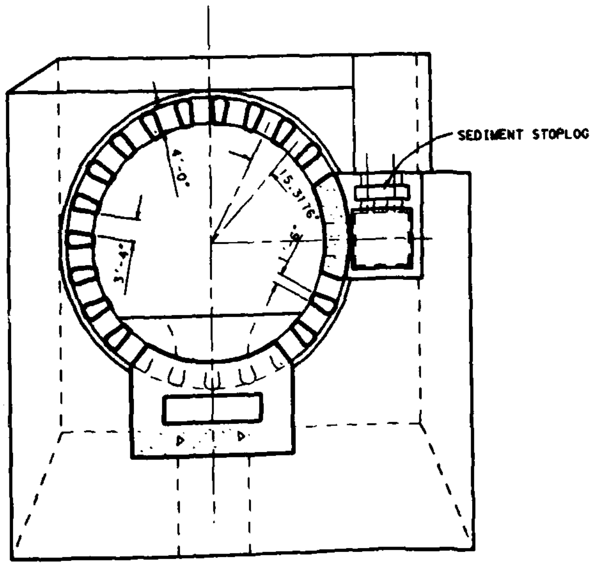

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SECTION D

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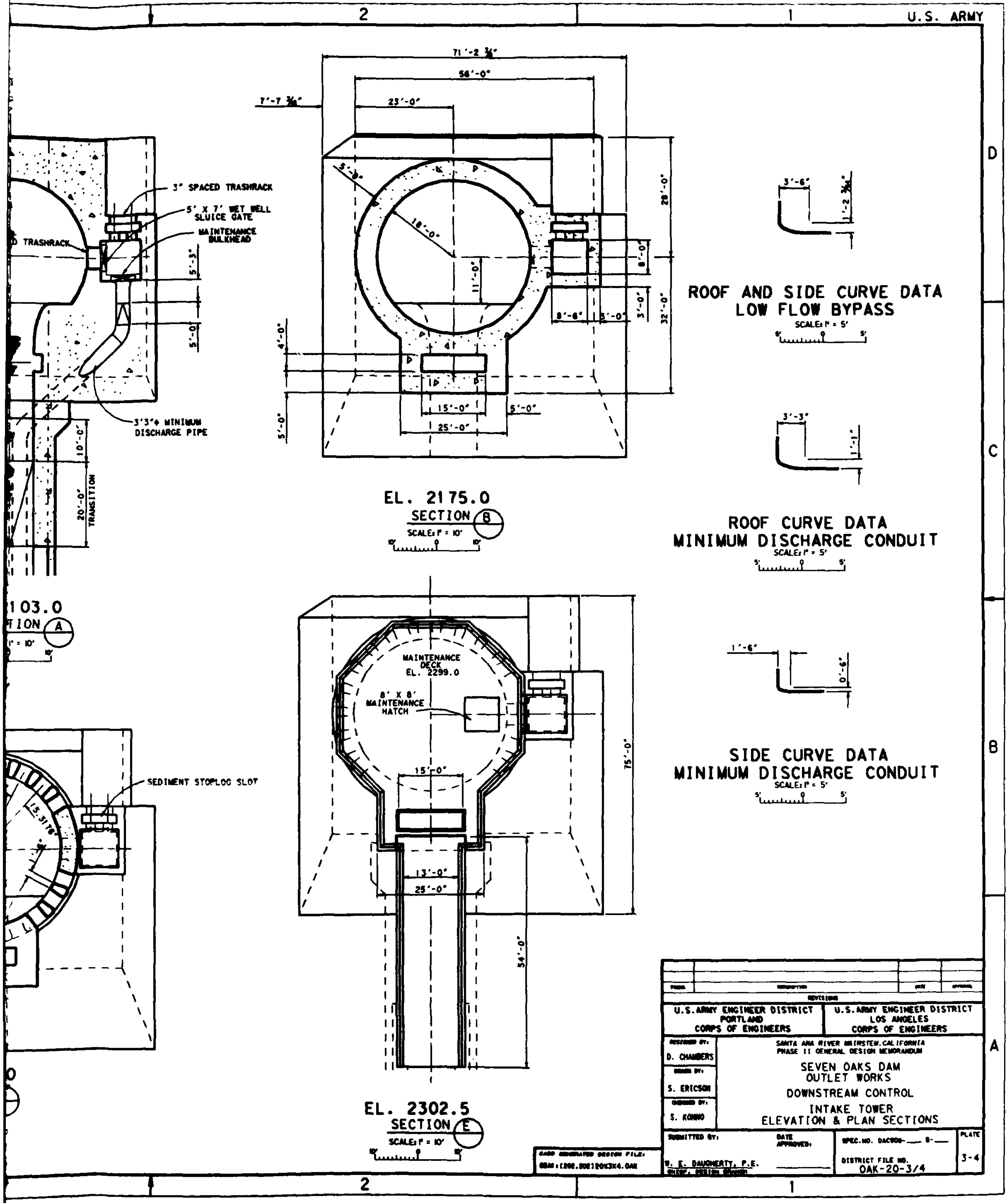
A

4 3



REVISIONS

U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS

U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS

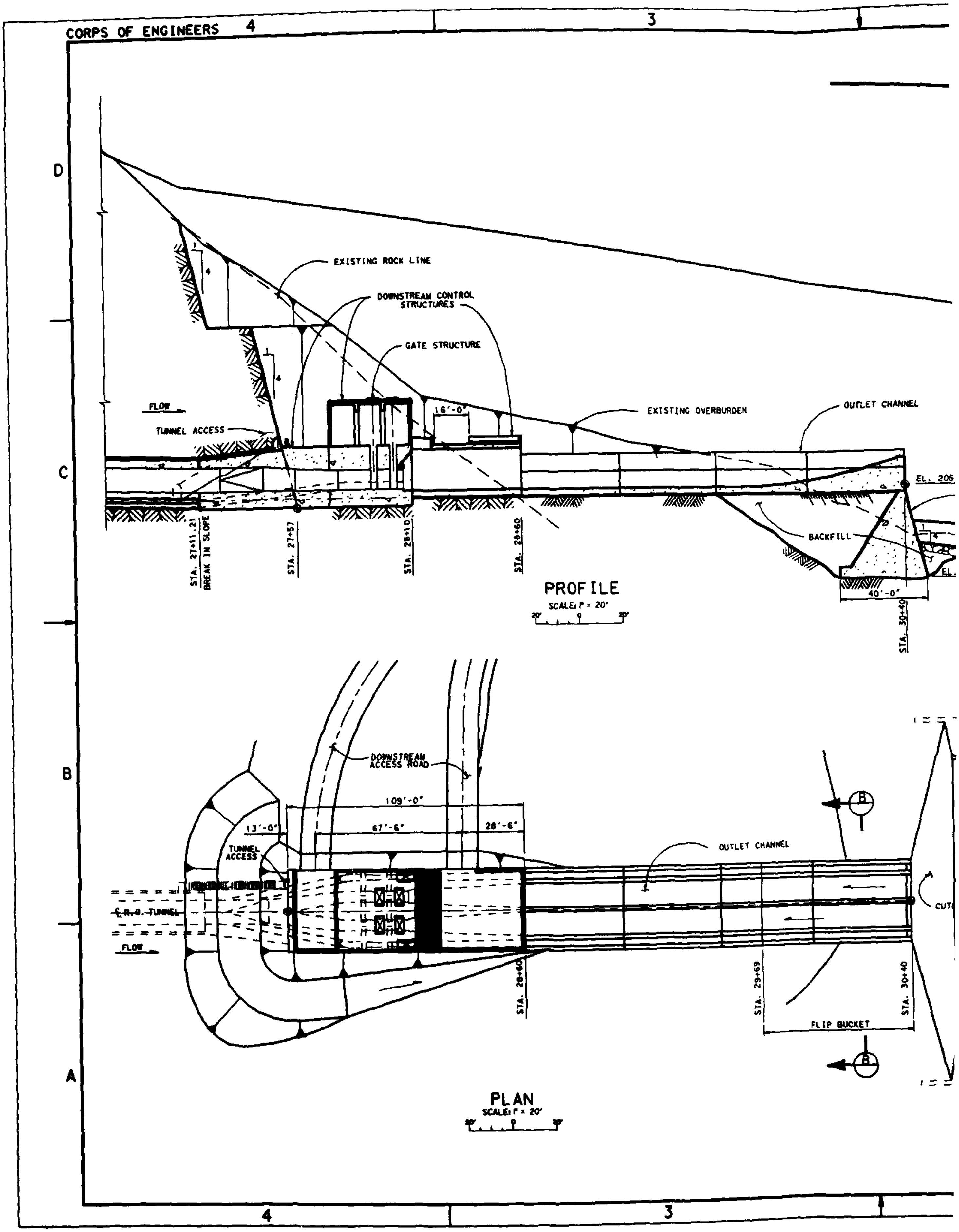
DESIGNED BY: D. CHAMBERS
DRAWN BY: S. ERICSON
CHECKED BY: S. KONNO

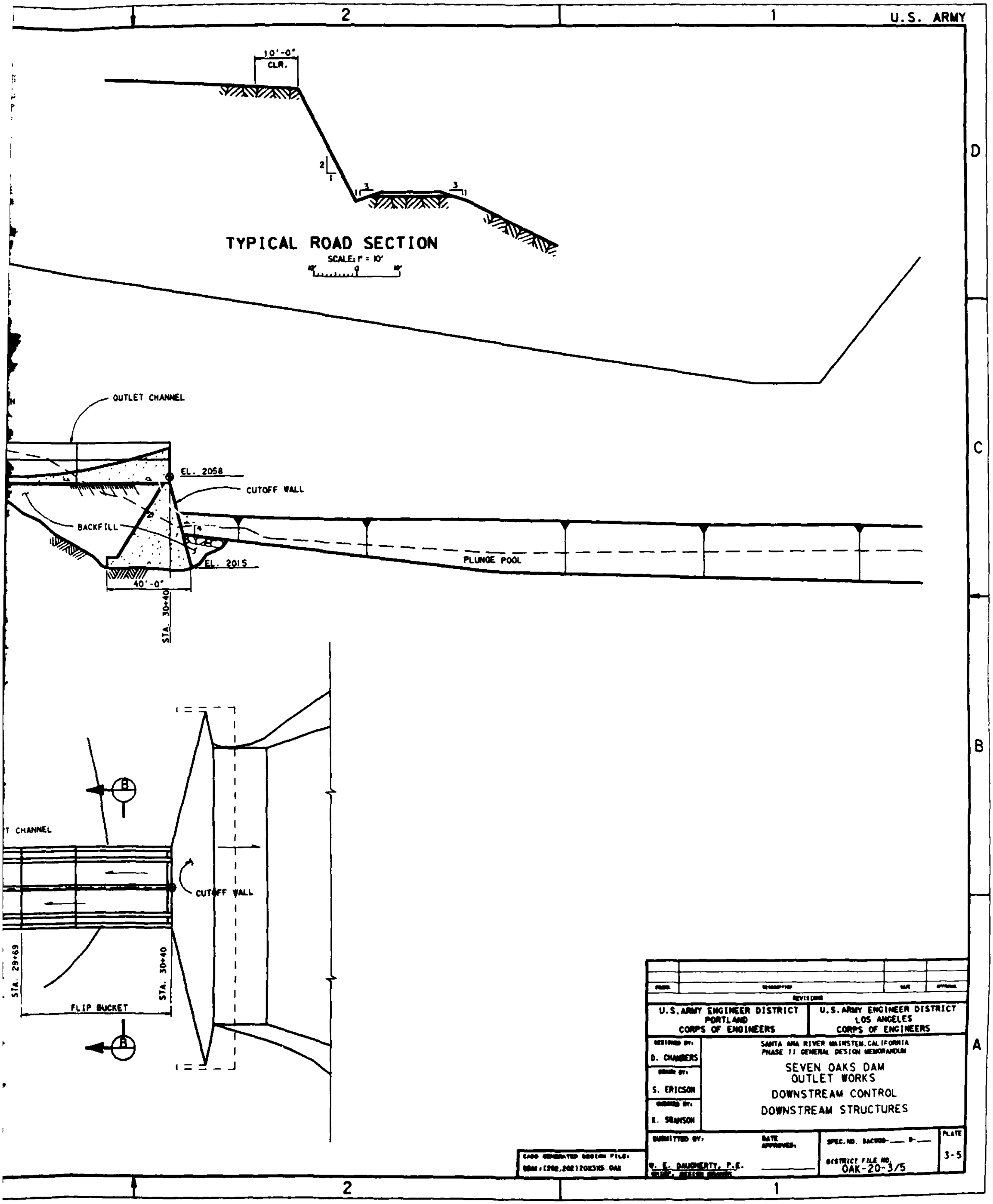
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
OUTLET WORKS
DOWNSTREAM CONTROL
INTAKE TOWER
ELEVATION & PLAN SECTIONS

SUBMITTED BY: W. E. DAUGHERTY, P.E. CHIEF, DESIGN BRANCH
DATE APPROVED:
SPEC. NO. DACW09-
DISTRICT FILE NO. OAK-20-3/4
PLATE 3-4

2
1


CORPS OF ENGINEERS
4
3
D
C
B
A
EXISTING ROCK LINE
DOWNSTREAM CONTROL STRUCTURES
GATE STRUCTURE
FLOW
TUNNEL ACCESS
16'-0"
EXISTING OVERBURDEN
OUTLET CHANNEL
EL. 205
BACKFILL
40'-0"
STA. 27+41.21 BREAK IN SLOPE
STA. 27+57
STA. 28+10
STA. 28+60
STA. 30+40
PROFILE
SCALE: 1" = 20'
DOWNSTREAM ACCESS ROAD
109'-0"
13'-0"
67'-6"
28'-6"
TUNNEL ACCESS
OUTLET CHANNEL
℄ R.O. TUNNEL
FLOW
STA. 28+60
STA. 29+69
STA. 30+40
FLIP BUCKET
B
PLAN
SCALE: 1" = 20'
4
3


U.S. ARMY
2
1
10'-0"
CLR.
TYPICAL ROAD SECTION
SCALE: 1" = 10'
OUTLET CHANNEL
EL. 2058
CUTOFF WALL
BACKFILL
PLUNGE POOL
EL. 2015
40'-0"
STA. 30+40
T CHANNEL
CUTOFF WALL
STA. 29+69
STA. 30+40
FLIP BUCKET
B
D
C
B
A
REVISIONS
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PORTLAND
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LOS ANGELES
CORPS OF ENGINEERS
D. CHAMBERS
S. ERICSON
K. SWANSON
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
OUTLET WORKS
DOWNSTREAM CONTROL
DOWNSTREAM STRUCTURES
SUBMITTED BY:
DATE APPROVED:
SPEC. NO.
PLATE
3-5
DISTRICT FILE NO.
OAK-20-3/5
W. E. DAUGHERTY, P.E.
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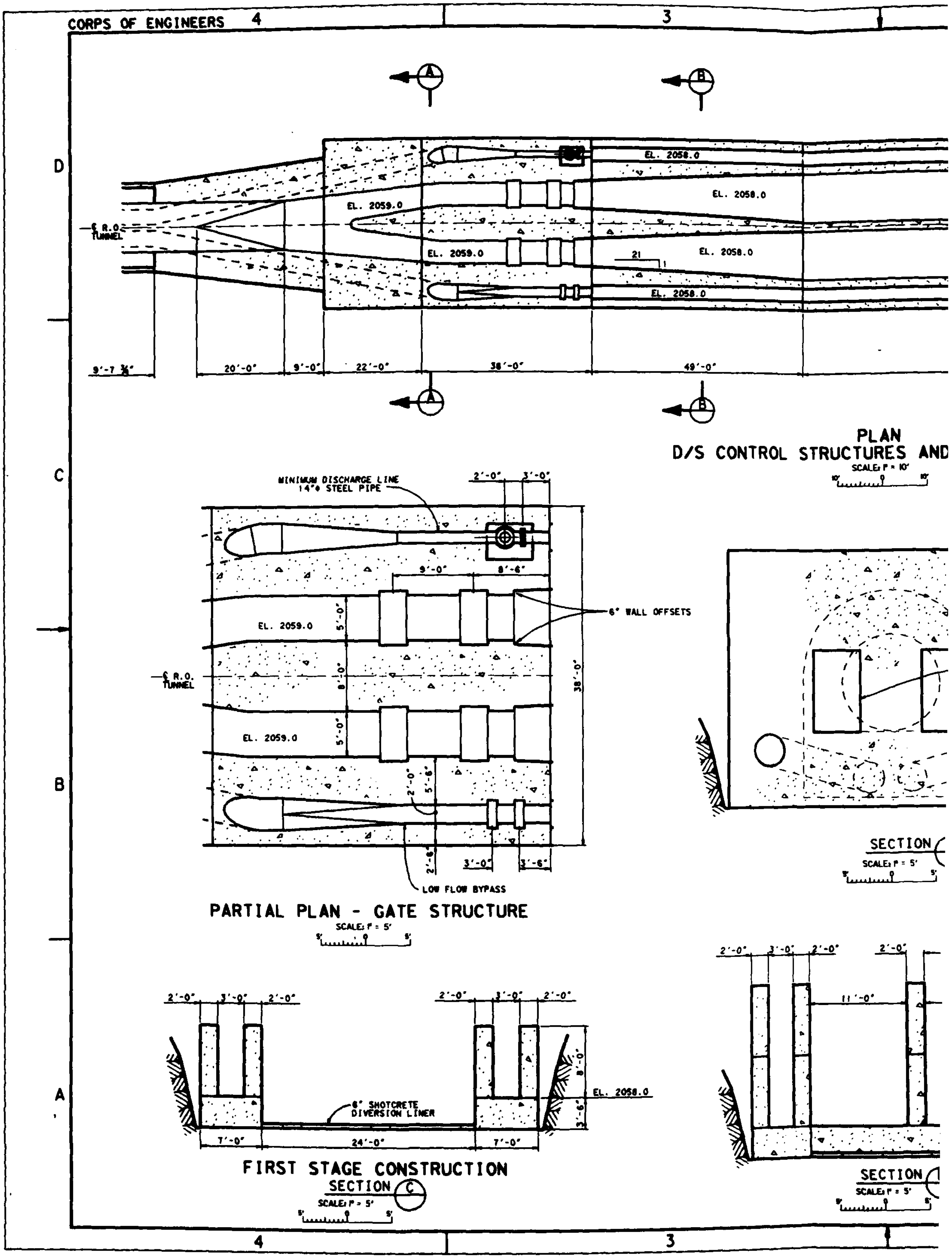
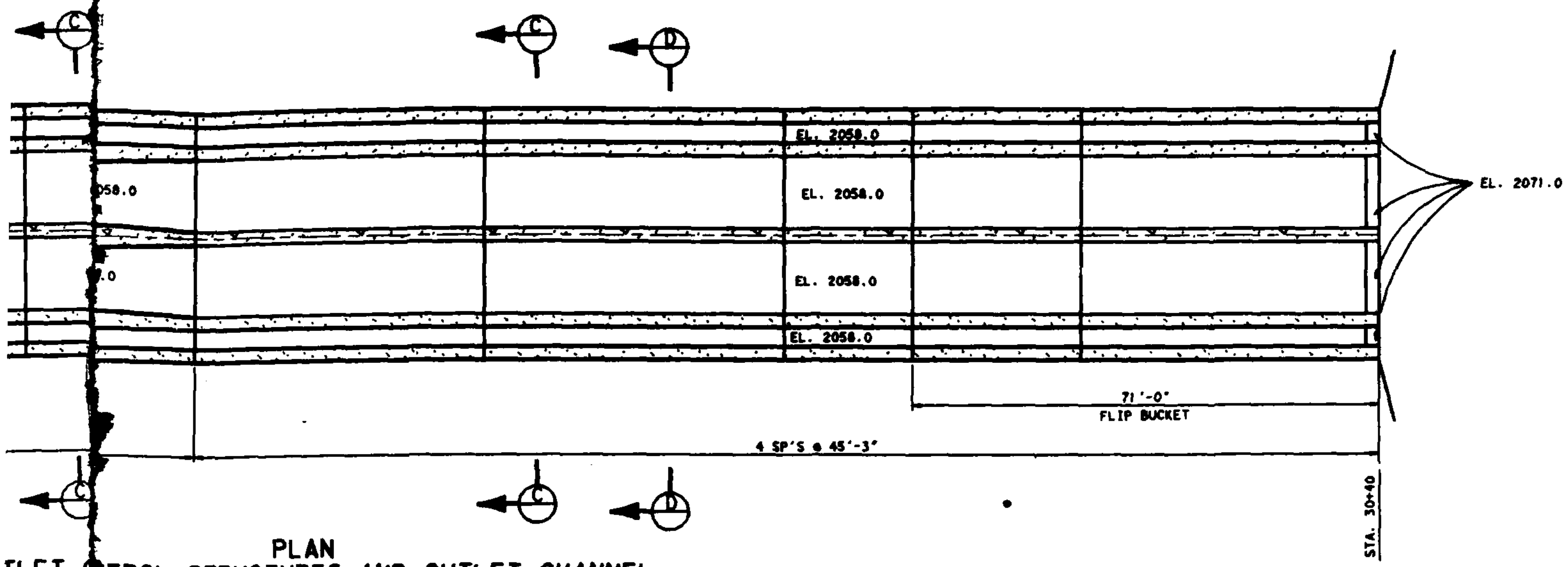
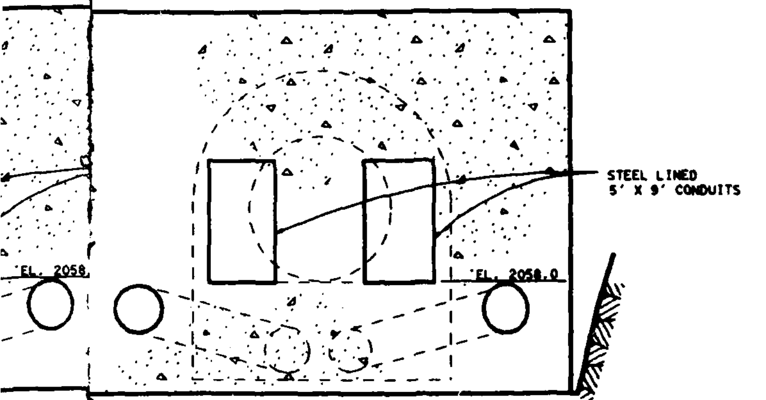


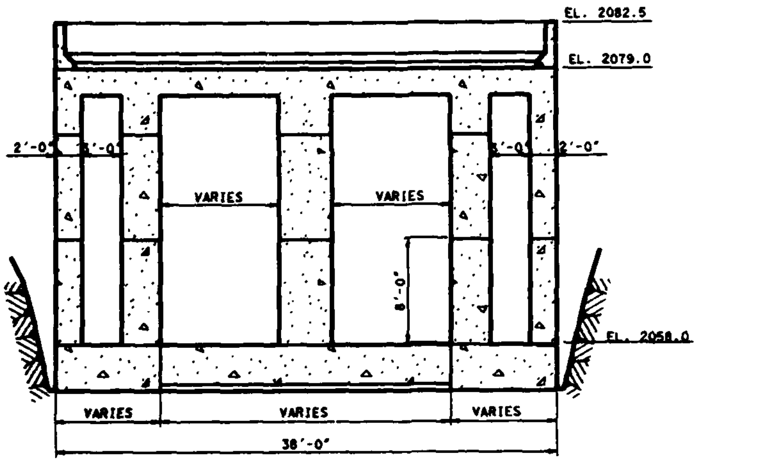
CORPS OF ENGINEERS
4
3
D
C
B
A
EL. 2058.0
EL. 2059.0
℄ R.O. TUNNEL
21
1
9'-7 ¾"
20'-0"
9'-0"
22'-0"
38'-0"
49'-0"
PLAN
D/S CONTROL STRUCTURES AND
SCALE: 1" = 10'
MINIMUM DISCHARGE LINE 14"⌀ STEEL PIPE
2'-0"
3'-0"
9'-0"
8'-6"
6" WALL OFFSETS
5'-0"
8'-0"
38'-0"
5'-6"
2'-6"
3'-6"
LOW FLOW BYPASS
PARTIAL PLAN - GATE STRUCTURE
SCALE: 1" = 5'
SECTION
SCALE: 1" = 5'
11'-0"
6" SHOTCRETE DIVERSION LINER
3'-6"
7'-0"
24'-0"
FIRST STAGE CONSTRUCTION
SECTION C
SCALE: 1" = 5'



PLAN
TLET [C]NTROL STRUCTURES AND OUTLET CHANNEL
SCALE: 1" = 10'



SECTION A
SCALE: 1" = 5'



SECTION B
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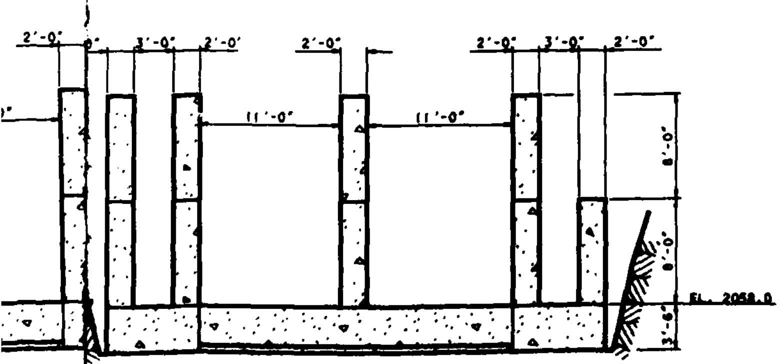

SECTION D
SCALE: 1" = 5'

CADD GENERATED DESIGN FILE:
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REVISIONS

U.S. ARMY ENGINEER DISTRICT
PORTLAND
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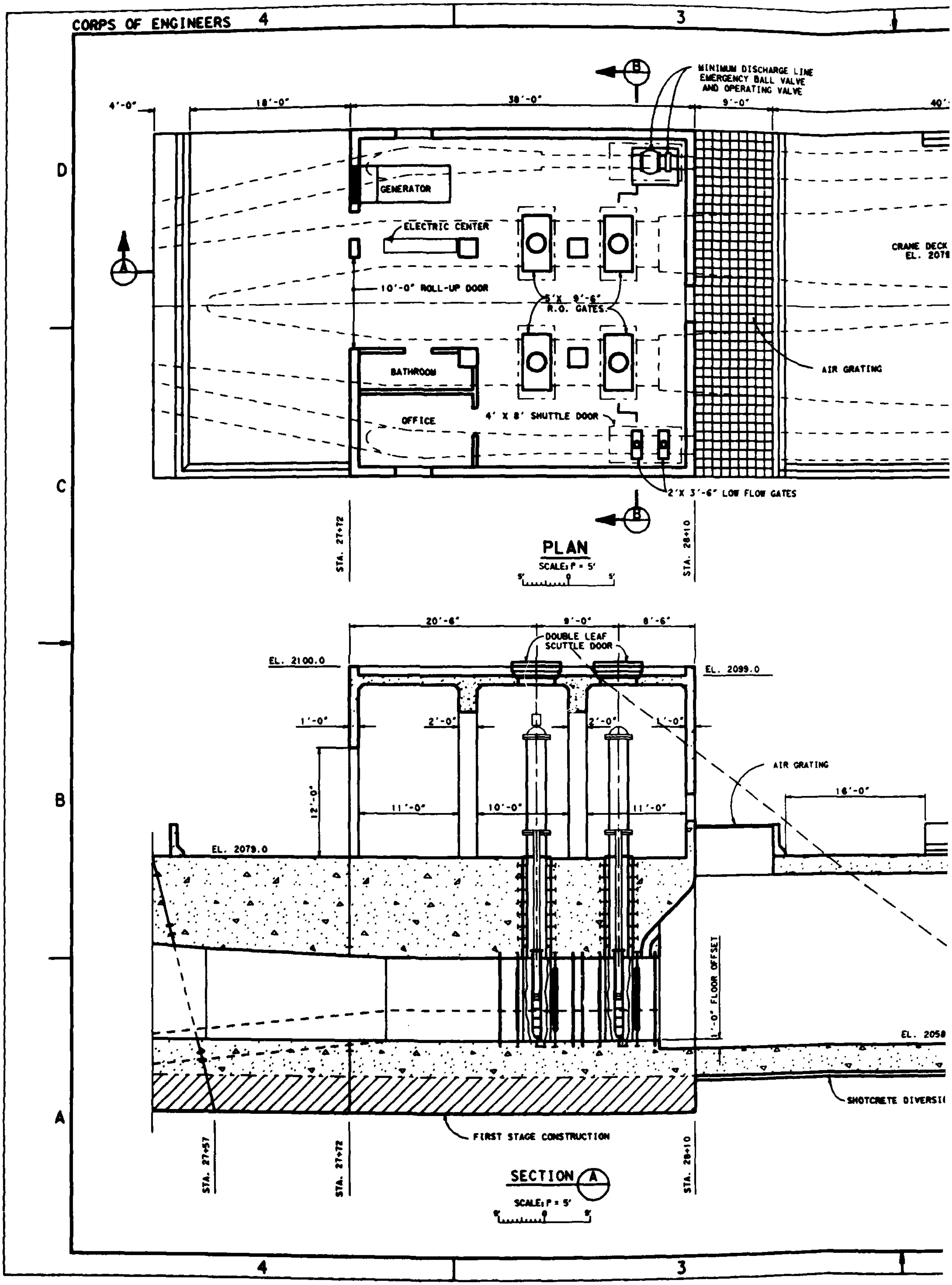
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: D. CHAMBERS
DRAWN BY: S. ERICSON
CHECKED BY: K. SWANSON

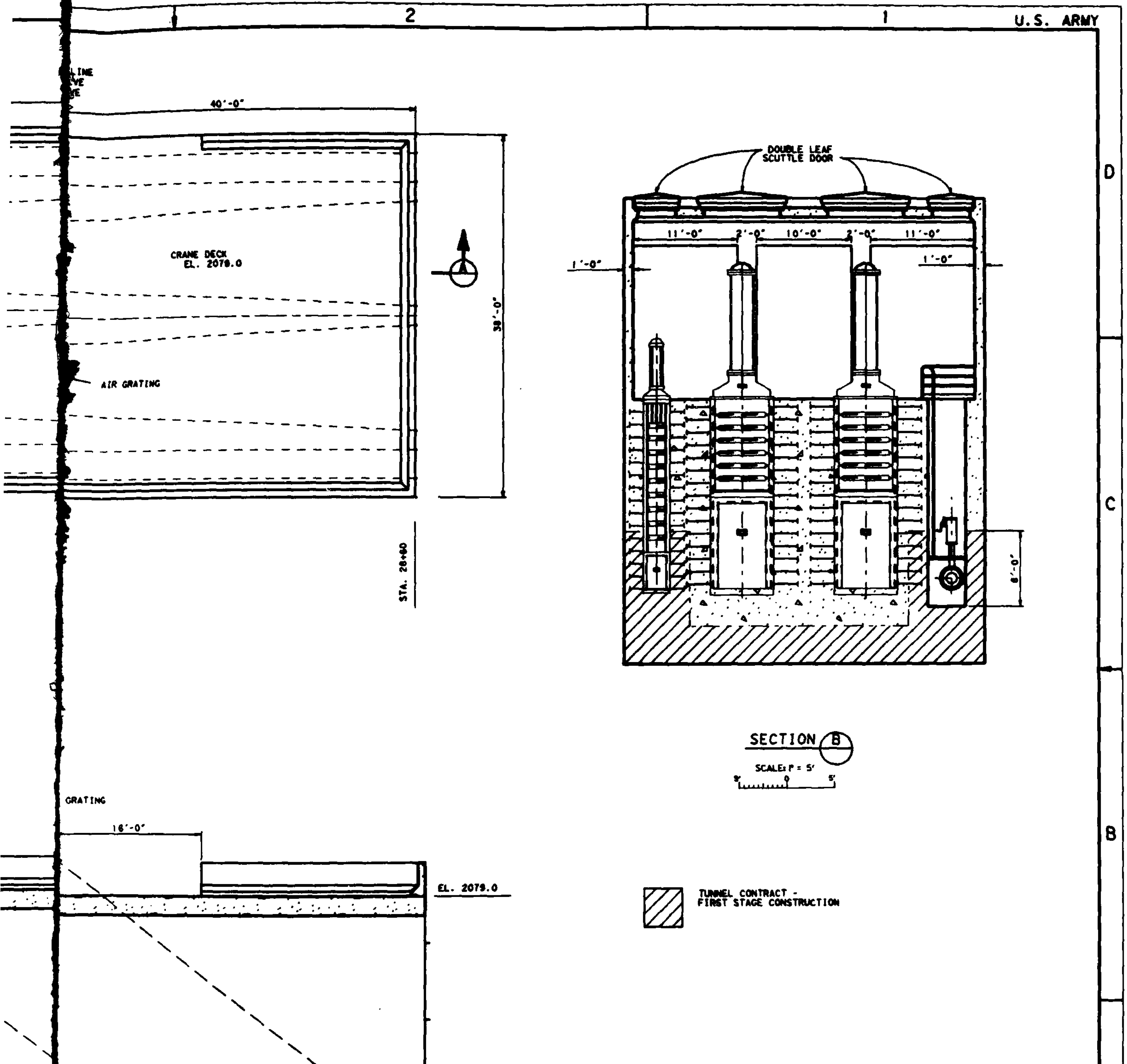
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
OUTLET WORKS
DOWNSTREAM CONTROL
DOWNSTREAM STRUCTURES
PLAN & SECTIONS SHEET 1

SUBMITTED BY: W. E. DAUGHERTY, P.E.
CHIEF, DESIGN BRANCH
DATE APPROVED:
SPEC. NO. DACW09-___ B-___
DISTRICT FILE NO. OAK-20-3/6
PLATE 3-6



CORPS OF ENGINEERS
4
3
MINIMUM DISCHARGE LINE
EMERGENCY BALL VALVE
AND OPERATING VALVE
4'-0"
18'-0"
38'-0"
9'-0"
GENERATOR
ELECTRIC CENTER
CRANE DECK
10'-0" ROLL-UP DOOR
5'X 9'-6"
R.O. GATES
AIR GRATING
BATHROOM
OFFICE
4' X 8' SHUTTLE DOOR
2'X 3'-6" LOW FLOW GATES
STA. 27+72
STA. 28+10
PLAN
20'-6"
9'-0"
8'-6"
DOUBLE LEAF
SCUTTLE DOOR
EL. 2100.0
EL. 2099.0
1'-0"
2'-0"
2'-0"
1'-0"
AIR GRATING
16'-0"
12'-0"
11'-0"
10'-0"
11'-0"
EL. 2079.0
1'-0" FLOOR OFFSET
SHOTCRETE DIVERSI
FIRST STAGE CONSTRUCTION
STA. 27+57
STA. 27+72
STA. 28+10
SECTION A



REVISIONS

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LOS ANGELES
CORPS OF ENGINEERS

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
OUTLET WORKS
DOWNSTREAM CONTROL
DOWNSTREAM STRUCTURES
PLAN & SECTIONS SHEET 2

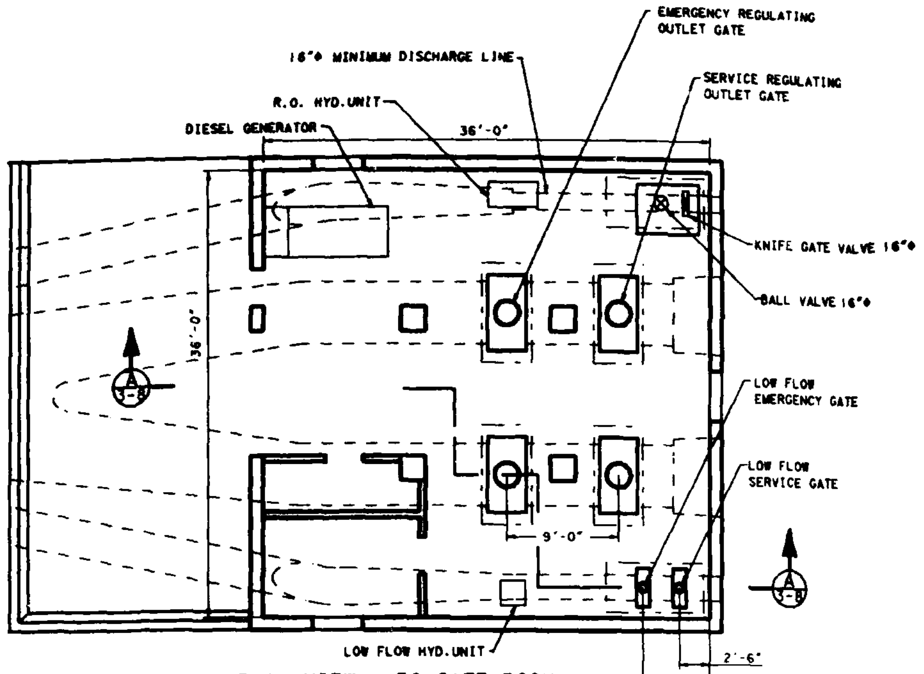
D. CHAMBERS

S. ERICSON

DISTRICT FILE NO.
OAK-20-3/7

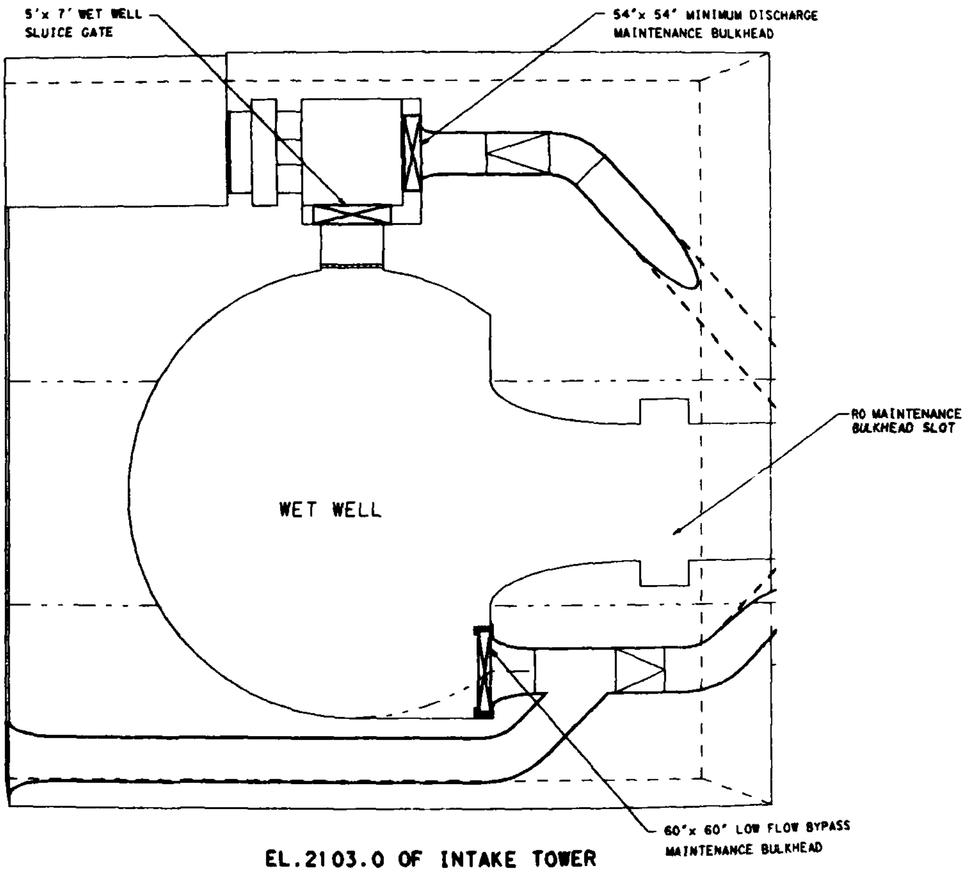
PLATE
3-7

CORPS OF ENGINEERS



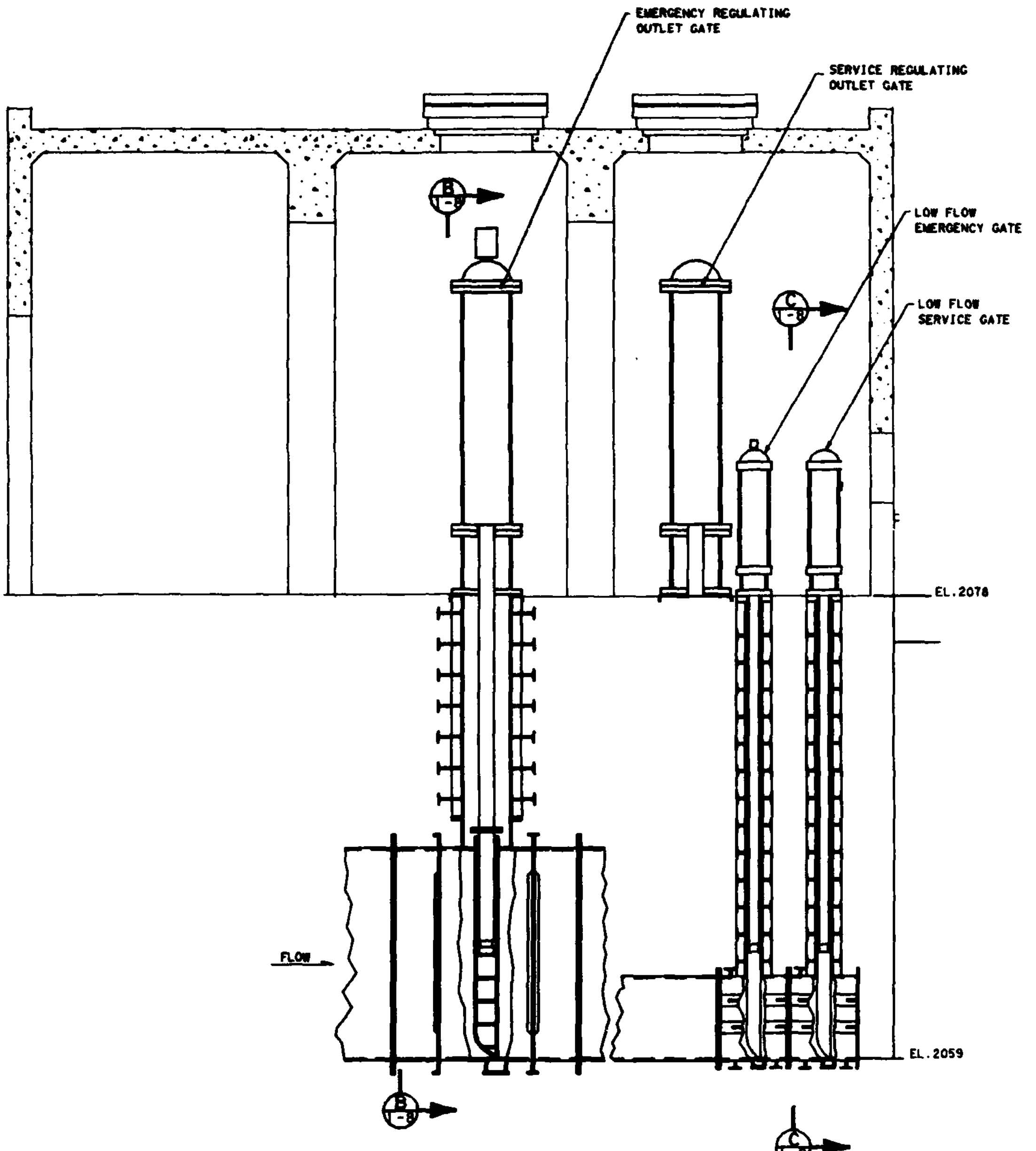
PLAN VIEW - RO GATE ROOM

SCALE: [illegible] = 1'-0"



EL. 2103.0 OF INTAKE TOWER

SCALE: [illegible] = 1'-0"



SECTION A 3-8

SCALE: 3/8" = 1'-0"

NOTES:

1. SEE DRAWING OAK-20-1/8 FOR FRONT ELEVATION OF SLIDE GATES AND HYDRAULIC SCHEMATIC DIAGRAM.

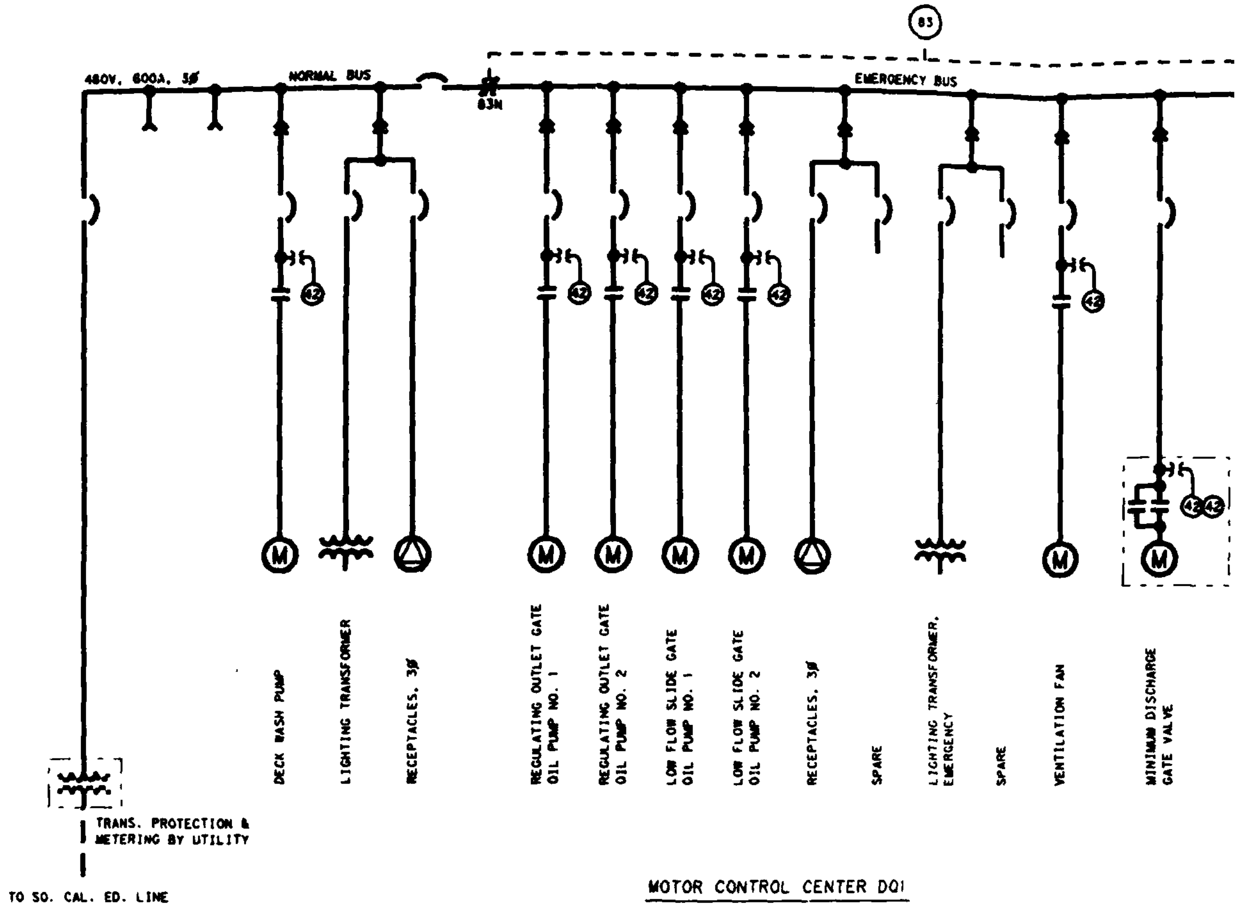
U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS	U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS

DESIGNED BY: KST	SANTA ANA RIVER MAINSTEM, CALIFORNIA PHASE II GENERAL DESIGN MEMORANDUM
DRAWN BY: DAB	SEVEN OAKS DAM OUTLET WORKS DOWNSTREAM CONTROL
CHECKED BY: RAM	MECHANICAL EQUIPTMENT LAYOUT

SUBMITTED BY: W.E. DAUGHERTY, P.E. CHIEF, DESIGN BRANCH	DATE APPROVED:	SPEC. NO. DACW09-__ B-__ DISTRICT FILE NO. OAK-20-3/8	SHEET 3-8

CADD DESIGN FILE: 0SM:(202,202)20C3X8.OAK

CORPS OF ENGINEERS 4 3

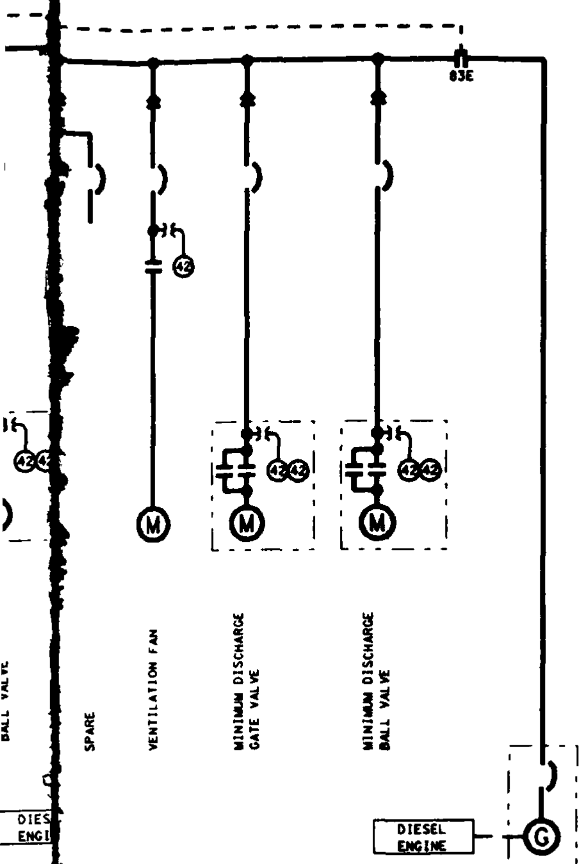


MOTOR CONTROL CENTER DQ1

D C B A

4 3

U.S. ARMY



LEGEND

- (G) GENERATOR
- (M) MOTOR
- RECEPTACLE, 3∅, 480V
- CIRCUIT BREAKER OR MOTOR CIRCUIT PROTECTOR
- MOTOR STARTER OR CONTACTOR
- 83 AUTO-TRANSFER SWITCH
  E-EMERGENCY
  N-NORMAL

REVISIONS

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SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
OUTLET WORKS
DOWNSTREAM CONTROL
ONE-LINE DIAGRAM

DESIGNED BY: RTO
DRAWN BY: JLH
CHECKED BY: KAB

SUBMITTED BY: D. E. DAUGHERTY, P.E.
DATE APPROVED:

SPEC. NO. DACW09- -
DISTRICT FILE NO. OAK-20-3/9
PLATE 3-9

SECTION 4

MID-TUNNEL CONTROL - OUTLET WORKS

4.1 General. This section describes the regulating outlet (RO) features for an outlet works with control gating located within the upstream rock mass (see profile, plate 4-2). The system is comprised of a high level intake tower, an RO diversion tunnel, an upstream segment of pressurized tunnel, and controls at approximately 435 feet downstream of the tunnel entrance with shaft access and tower. An open-flow channel tunnel continues downstream of the control structure with an outlet channel connecting the tunnel portal to an energy dissipating flip bucket and plunge pool.

4.2 Intake.

a. General. The intake tower is a reinforced concrete structure with a maximum height of 222.5 feet partially embedded into a dioritic rock formation (see plate 4-3). The lower 76 feet of the tower is surrounded by rock on three sides. The structure cantilevers above El. 2,156 as a freestanding tower for 146.5 feet. The tower height was set based on an expected sediment deposition over the project life of 165 feet, or from El. 2,100 to El. 2,265. The sill of the intake tower for floodflows (normal operating maximum of 8,000 cubic feet per second [cfs]) is located at the 2,265 elevation. The tower is designed for operation under submerged conditions. The top 146.5 feet of the tower is essentially a light circular structure with a maintenance deck and access bridge at El. 2,299. Below El. 2,156, the tower is characterized by a more massive rectangular section which houses the multilevel withdrawal wet well, RO wet well, provisions for maintenance gating, and minimum discharge entrance and piping. Below El. 2,265, on the left side of the tower, is the multilevel withdrawal system. This system is a series of small diameter intakes used to regulate the lower debris pool. Outflow from the withdrawal system discharges into the large 36-foot-diameter tower wet well. The debris pool after 100 years of reservoir

sedimentation or a flood event at year zero will see the tower submerged, while a standard project flood (SPF) will submerge the tower by approximately 280 feet.

b. Approach Channel. For the first operating years, the diversion approach channel will become the operating RO intake channel. The channel will be approximately 30 feet wide at the bottom, with alluvial side slopes of 1V on 2H. The elevation of the channel invert at the tower will be 2,100 rising back naturally to the river channel upstream of the dam site at an approximate channel length of 1,040 feet and at a 3 percent slope.

c. Access. Tower access to the maintenance deck is required to perform yearly inspection, maintenance, bulkheading, and debris handling. The tower will be accessible from a road off of the top of the embankment dam from the right abutment across the upstream dam face, to the left abutment where it is cut into rock, to a point immediately downstream of the tower. The final access leg is accomplished by a 60-foot span single-lane bridge to the El. 2,299 maintenance deck. The road is approximately 4,800 feet long, sloped at an average of 6 percent, single lane (with turnouts), and paved. The road will be designed for project crane and dump truck operations. Longer bridges were investigated to minimize rock excavations, but preliminary analysis found associated pier heights to be prohibitive in this seismic environment. Other road alignments on the left reservoir were found to have excessive length and rock excavation. There was also the Southern California Edison penstock crossing to consider.

d. High Level Intake. The primary intake is designed to pass the regulated higher flows in accordance with the operating criteria as depicted in section 6. Upwards of 8,000 cfs can be passed as a normal design flow through the 122 3-foot 4-inch-square openings covering an area from El. 2,265 to El. 2,292.5. The openings are separated on all sides by 1-foot 6-inch beams (trash struts). Openings extend 315 degrees around the circumference of the intake tower. Eight hundred square feet are required for entrance velocity conditions; an additional 556 square feet have been provided as a safety factor against plugging. Flow past the trash struts enters a 36-foot-diameter wet well. In the ceiling of the

intake is an 8-foot-square access hatch, provided for inspection, cleanout, and any maintenance required.

e. Wet Well. Within the tower is a 36-foot-diameter circular wet well. It extends for the full tower height, some 195 feet. The walls of the tower are constructed on top of a 20-foot-thick massive rectangular base slab. The walls are mass concrete from the base at El. 2,100 up to El. 2,156. In plan view, the tower below El. 2,156 is rectangular on the outside. Above El. 2,156, the tower is circular with a 5-foot-thick wall reducing to 4 feet for the top 84.5 feet. Near the bottom of the tower the downstream face of the wet well flattens to form a vertical plane where the square bell-mouth entrance to the outlet conduit is found (see plate 4-3). Within the upstream face of the tower at El. 2,100, is a 5-foot-diameter conduit which will be used to facilitate maintenance at the intake structure.

f. Regulating Outlet. The main RO will handle the larger flows, primarily flows above 300 cfs under normal operating conditions. The entrance to the RO conduit is found at the bottom of the large wet well. The opening is an 18-foot horseshoe opening. The RO will have a bulkhead slot located at the RO entrance. The maintenance bulkhead would be stored at the bridge abutment. The bulkhead will be a slide gate capable of being lowered by crane under submerged conditions of a reservoir elevation less than the maintenance deck at El. 2,299. Guides for the bulkhead are located on the interior of the RO wet well (see plage 4-3).

g. Multilevel Withdrawal System. The multilevel withdrawal system (MWS) consists of a single column of 3-foot 8-inch-diameter ports spaced at 10 vertical feet for the full height of the sediment range (El. 2,100 to El. 2,265). The ports are located on the left side of the tower. This system regulates the reservoir storage area below the intake tower sill at El. 2,265. The MWS ports are covered with trashrack grating and discharge into an 8-foot 6-inch by 8-foot wet well. An access hatch for maintenance will be located at the top of the wet well. The wet well discharges into the larger 36-foot wet well through a 5- by 7-foot conduit at El. 2,100. A large manually operated wet well sluice gate will be

located at the conduit entrance with an operator located above El. 2,265. This gate will not be used as a throttling gate, because it will be used either fully opened or fully closed. A stoplog slot is located between the trashracks and wet well. Prior to flood season, sediment stoplogs will be installed. The concrete stoplogs will be placed in advance of the rising sediment (approximately 20 to 30 feet above the sediment level to account for the predicted rise in sediment depth from an SPF event). With the stoplogs utilized, sediment passage through the project will be minimized.

h. Minimum Discharge Line. Flows between 10 and 100 cfs will be passed through a minimum discharge line (24-inch pipeline) originating at the bottom of the MWS wet well (see plate 4). These flows cannot be accommodated by the hydraulic slide gates between pool Els. 2,100 and 2,350, see figures 6-1 and 6-2. Upstream bulkheading slots will be provided for maintenance. The minimum discharge line transitions to a 14-inch line within the mid-tunnel control structure. Control of flow is achieved using a 14-inch valve with an emergency ball valve for backup. An alternative design is being considered to provide a 3-foot-diameter pressure pipe to carry flow to the downstream end of the outlet works. Flow would be regulated at the downstream end by a cone valve (see plate 5/6).

4.3 Regulating Outlet/Diversion Tunnel.

a. General. The outlet tunnel is 1,647 feet in length and will be constructed as a horseshoe tunnel section utilizing conventional drill and blast techniques (see plate 4-2). The dimensions of the tunnel will be 18 feet wide and 18 feet high within the inside face of the concrete. An 18-foot-wide blockout within the control structure will allow passage of diversion flows during the embankment construction. Once the embankment dam nears the standard project flood height, the mid-tunnel gating will be installed in the diversion tunnel. Gate installation will take place during the low flow months when the handling of water will be through the minimum discharge unit or staged through either side of the RO conduit blockout. The tunnel has a constant slope of 0.026.

b. Zone A, Station 11+30 to Station 15+08. The first 328 feet of the tunnel is a pressurized 18-foot horseshoe with thick heavily-reinforced concrete walls able to withstand the external and internal loads expected for this zone. The remaining 50 feet contain a 25-foot-transition to a 23-foot-wide horseshoe followed by a 25-foot-straight section prior to entering the 5- by 9-foot RO conduits within the control monoliths.

c. Zone B, Station 15+08 to Station 16+90. The mid-tunnel control is comprised of five sections. The first section is 28 feet in length and houses the entrances for the low-flow bypass and the two RO conduits. The next section is the main mid-tunnel control monolith. This section is 57 feet long and contains all the regulating gating and associated equipment and shaft access. This section is a large horseshoe, 45 feet wide by 59 feet high. The remaining downstream monoliths total 104 feet in length. The sections are horseshoe, approximately 34 feet wide by 28 feet high. The sections have the conduit transitions with splitter walls (see plate 4-2).

d. Zone C, Station 16+90 to Station 27+57. This tunnel section contains a 20-foot straight section with an inside width of 24 feet. The tunnel then transitions over 65 feet to the typical 18-foot-wide open-flow horseshoe section which extends for 982 feet until it exits at the downstream portal.

4.4 Mid-Tunnel Control. The mid-tunnel control structure is located approximately at the first quarter point from the upstream portal, between Stations 15+35 and 15+86. This structure houses the RO conduits, minimum discharge system, hydraulic slidegates, gate room, and the interception with air and access shafts.

a. Regulating Outlets. The entrances to the outlet conduits are found within the tunnel upstream of the control monolith. There are three bell-mouth openings which reduce to two 5- by 9-foot and one 2- by 3 1/2-foot conduits (see plate 4-5). The smaller opening will have a steel trashrack at the face sized for the smaller conduit (16-inch bar spacing). Maintenance bulkheading for these conduits will be accomplished at the intake tower.

b. Minimum Discharge System. The minimum discharge line will transition from 24 inches to 16 inches just upstream of the valve pit. An in-line disc valve will be used for primary control with a ball valve for emergency usage. Flows will discharge into an 18-inch by 24-inch conduit which transitions into a 12-inch by 24-inch section. The conduit then extends downstream, enclosed in one of the separator piers as shown on plate 4-5.

c. Hydraulic Slidegates. The two 5- by 9-foot operating slide gates are located 50 feet downstream of the outlet conduit intakes. Emergency slide gates are located 8 feet upstream of the operation gates (see plates 4-5 and 4-8). Smaller 2-foot by 3 1/2-foot low flow gates are located as shown on plate 4-5. The low flow entrance will have a trashrack sized for two-thirds by the least conduit dimension equal to a maximum opening of 16 inches. The 2-foot by 3 1/2-foot low flow gates are required in conjunction with the minimum discharge line. The low flow gates discharge lower range flows up to 500 cfs. The minimum discharge valve releases flows of between 10 and 100 cfs. Immediately downstream of the operating gate, air is introduced by use of aeration offsets located about the perimeter of the conduit. This air is brought to the conduits by way of a vertical 10-foot-diameter air shaft.

d. Gate Room. A 41-foot by 34-foot gate room with a 30-foot-high ceiling is provided to house the hydraulic slide gates, and mechanical and electrical operating equipment. Overhead hoists are provided for maintenance. The room is sized primarily for the handling of gate components and house the aforementioned project equipment (see plates 4-5 and 4-6).

e. Access and Air Shafts.

(1) Access Shaft. An 18-foot OD, 411-foot access shaft is required to provide elevator and stairway access, mechanical and electrical conduits, and equipment removal. The shaft will be accessed by means of the control tower located above El. 2,550 at the top of the shaft. The shaft will have a reinforced and drained concrete lining.

(2) Air Shaft. The air shaft is primarily a 10-foot ID shaft with a shotcreted and drained lining. The shaft will transition from 10-foot circular to a 6-foot by 12-foot rectangular section which bifurcates to provide air to the separate RO conduits (see plate 4-5). A 60-foot tower will be required to put the air intake above the high pool level.

(3) Shaft Alternative. A single shaft containing elevator, equipment and man access, and air supply was considered (see plate 4-5). This shaft was sized at 24-feet in diameter. It required more excavation and concrete than the two smaller shafts. The dual shaft concept also allows for the potential of using raised bore mining technology. The single shaft has an awkward air transition in the roof of the control structure. By itself, the single air shaft does not require a lining or drains whereas when combined into one shaft, total material volumes are increased. The material volumes increase because the larger shaft is lined, most likely drained, and designed for the expected external hydrostatic loads.

4.5 Control Tower. At the top of the access shaft, an 80-foot control tower will be required to put the entrance above the expected high water level. This tower will house the elevator machinery, gating controls, and related project equipment (diesel generator, reservoir readouts, control annunciation, HVAC, etc.), restroom, office, and storage. An estimated 60-foot access bridge will be required. A roll-up door is provided to access project equipment. Large gate components will be removed by crane through access hatches located in the roof.

4.6 Downstream Outlet Structures.

a. Exit Channel, Station 27+57 to Station 29+70. Upon exiting the tunnel, flows pass through 213 feet of concrete U-channel prior to being flipped and discharged into a plunge pool. The channel is 18 feet wide with 16-foot sidewalls (see plate 4-7).

b. Flip Bucket, Station 29+70 to Station 30+40. At the end of the outlet channel, a flip bucket will be provided to discharge the outflows from the two main regulating outlets. The flip bucket will be approximately 70 feet long and 20 feet high at the lip from the channel invert, with a radius of 120 feet. (See Hydraulics Section of GDM for further details.) Further refinements to the flip bucket trajectory angle and alignment will be investigated in the FDM level to minimize the outlet channel length, plunge pool configuration, and impacts to the downstream access road to the outlet works.

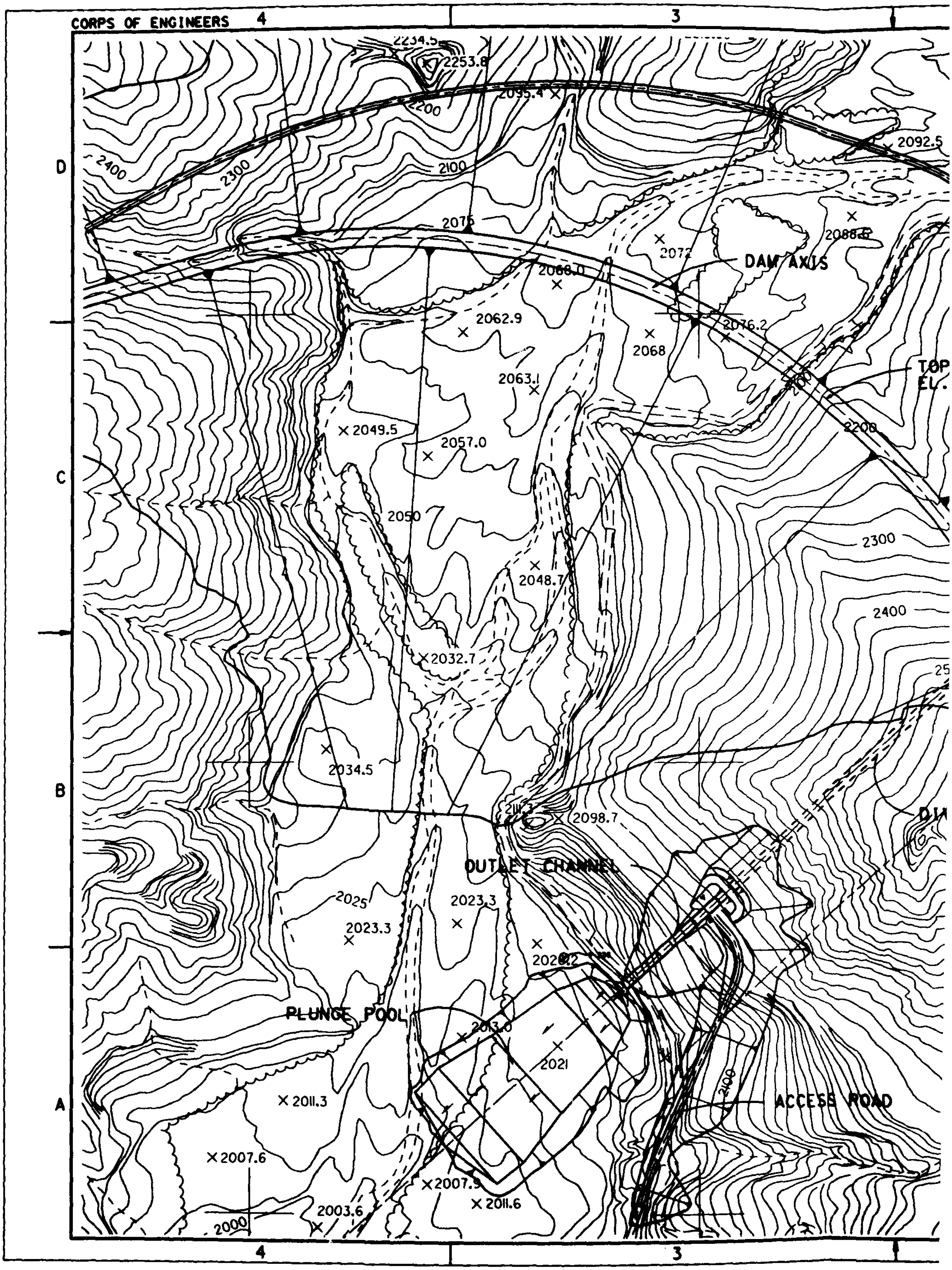
c. Cutoff Wall. The outlet channel terminates approximately 283 feet downstream of the tunnel portal. A near vertical faced cutoff wall is assumed required. At this point a cutoff gravity wall has been placed beneath the flip bucket. The cutoff wall protects against upstream undercutting of the outlet channel structures. A gravity wall has been utilized for its inherent abilities to withstand abrasion and a wide base is provided to minimize bearing, sliding, and settlement problems.

d. Plunge Pool. A scour hole will be pre-excavated to two-thirds the depth of the hole that theoretically would form from a constant 8,000 cfs release. This depth will provide initial energy dissipation to protect structures in the downstream channel until the scour hole reaches its equilibrium depth. The estimated final scour hole is a sufficient distance downstream that it will not pose any danger to the dam embankment. It is approximately 740 feet long and varies from 120 feet to 290 feet wide at the bottom with slide slopes of 1V on 3H. Larger rock will be left in the basin to accelerate the armoring process (see plate 4-8).

e. Downstream Access Road. The downstream access road will be a single-lane, paved road traversing the left abutment as shown (see general plan).

4.7 Instrumentation. The tower and downstream structures will be typically tied into the project survey monitoring program with key points identified by embedded monuments in the concrete. Tiltmeters and

extensometers will be utilized. A seismic accelerograph will be located in the gate room. Hydraulic instrumentation about the slide gates and immediately downstream will be utilized to monitor pressure and flow conditions. A specific plan and types of instrumentation should be developed at the FDM level.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2253.8
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2076.2
TOP
EL.
2063.1
2049.5
2057.0
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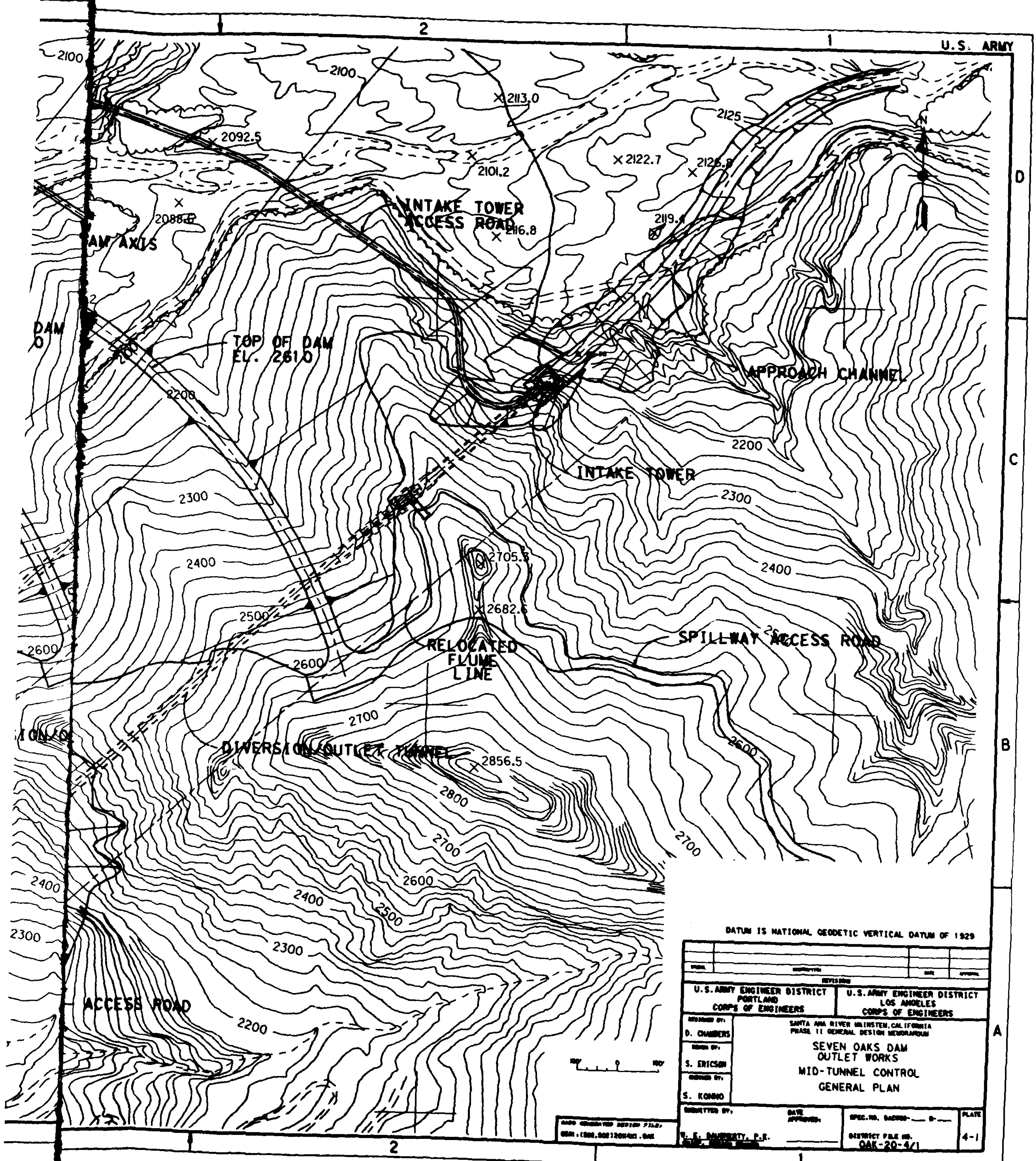
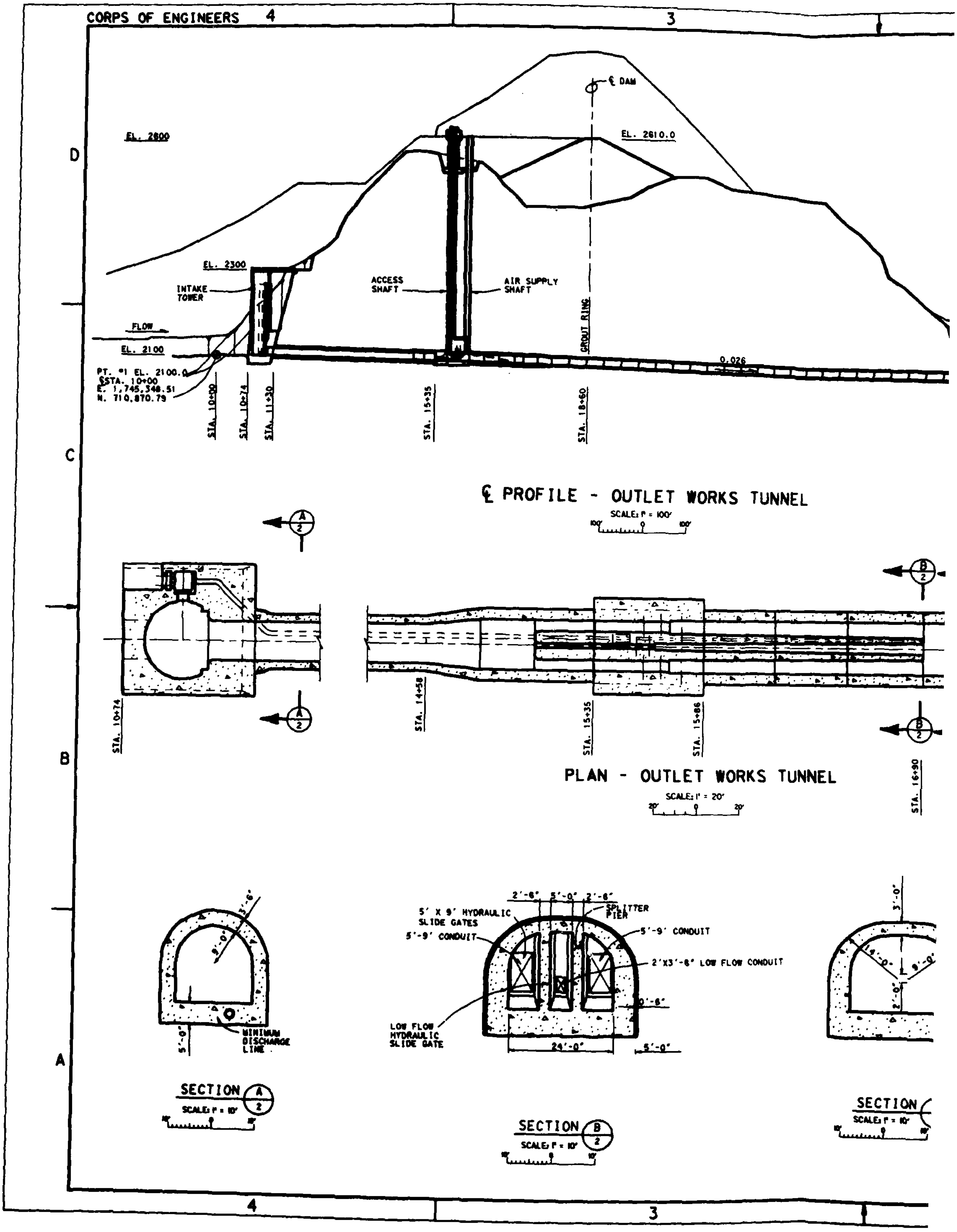
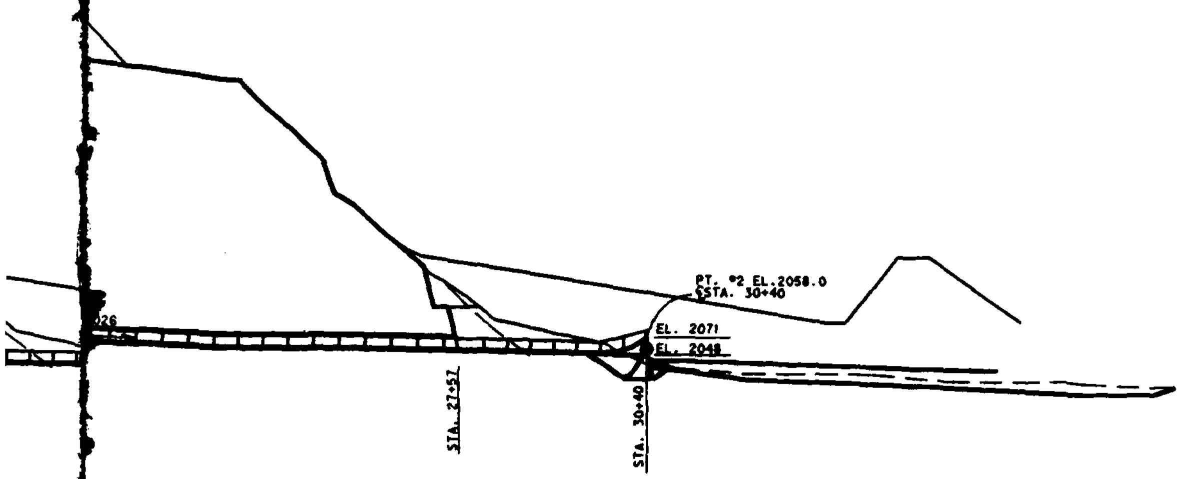
2050
2300
2048.7
2400
2032.7
2034.5
2111.3
2098.7
OUTLET CHANNEL
2025
2023.3
2023.3
2020.2
PLUNGE POOL
2013.0
2021
2100
ACCESS ROAD
2011.3
2007.6
2007.5
2011.6
2003.6
2000



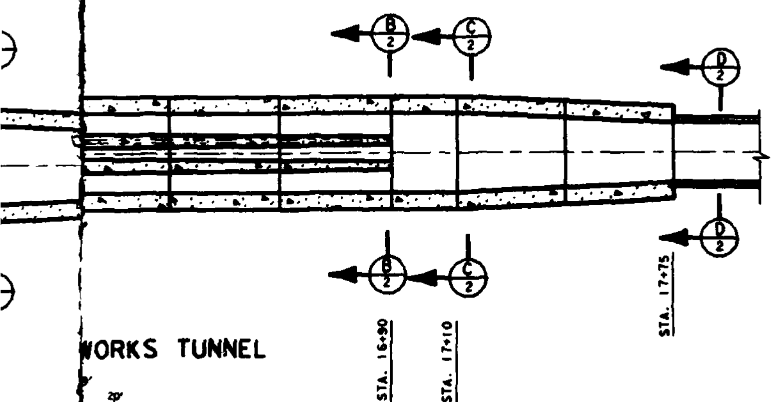
U.S. ARMY
INTAKE TOWER ACCESS ROAD
DAM AXIS
TOP OF DAM EL. 2610
APPROACH CHANNEL
INTAKE TOWER
SPILLWAY ACCESS ROAD
RELOCATED FLUME LINE
DIVERSION/OUTLET TUNNEL
ACCESS ROAD
2113.0
2092.5
2101.2
2122.7
2125
2119.4
2116.8
2705.3
2682.6
2856.5
2100
2200
2300
2400
2500
2600
2700
2800
DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929
U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS
U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS
D. CHAMBERS
S. ERICSON
S. KONNO
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
OUTLET WORKS
MID-TUNNEL CONTROL
GENERAL PLAN
DISTRICT FILE NO. OAK-20-4/1
PLATE 4-1


CORPS OF ENGINEERS
4
3
D
C
B
A
EL. 2600
℄ DAM
EL. 2610.0
EL. 2300
INTAKE TOWER
ACCESS SHAFT
AIR SUPPLY SHAFT
GROUT RING
FLOW
EL. 2100
PT. #1 EL. 2100.0
℄STA. 10+00
E. 1,745,348.51
N. 710,870.79
0.026
STA. 10+00
STA. 10+74
STA. 11+30
STA. 15+35
STA. 18+60
℄ PROFILE - OUTLET WORKS TUNNEL
SCALE: 1" = 100'
STA. 10+74
STA. 14+58
STA. 15+35
STA. 15+86
STA. 16+90
PLAN - OUTLET WORKS TUNNEL
SCALE: 1" = 20'
3'-6"
9'-0"
5'-0"
MINIMUM DISCHARGE LINE
SECTION A 2
SCALE: 1" = 10'
2'-6"
5'-0"
2'-6"
5' X 9' HYDRAULIC SLIDE GATES
SPLITTER PIER
5'-9' CONDUIT
5'-9' CONDUIT
2'X3'-6" LOW FLOW CONDUIT
LOW FLOW HYDRAULIC SLIDE GATE
24'-0"
5'-0"
SECTION B 2
SCALE: 1" = 10'
3'-0"
SECTION
SCALE: 1" = 10'

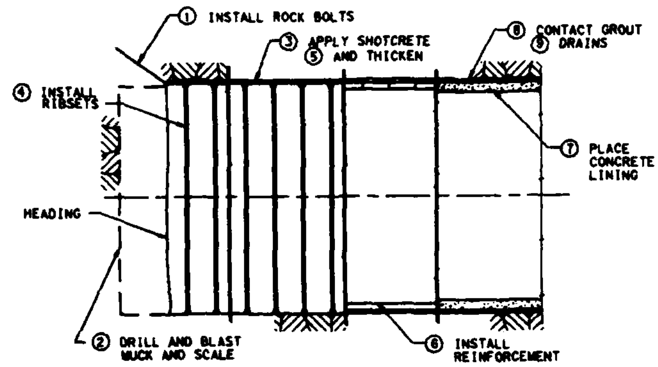
2 | 1 | U.S. ARMY



S TUNNEL



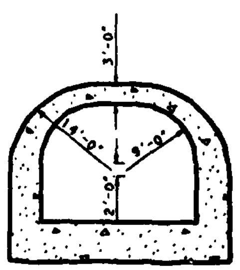
WORKS TUNNEL



TUNNELING SEQUENCE

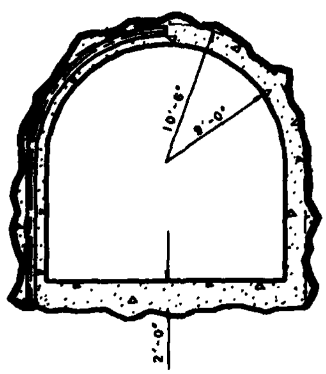
N.T.S.

FLOW CONDUIT



SECTION C/2

SCALE: 1" = 10'



SECTION D/2

SCALE: 1" = 5'

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D. CHAMBERS
S. ERICSON
K. SWANSON

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
OUTLET WORKS

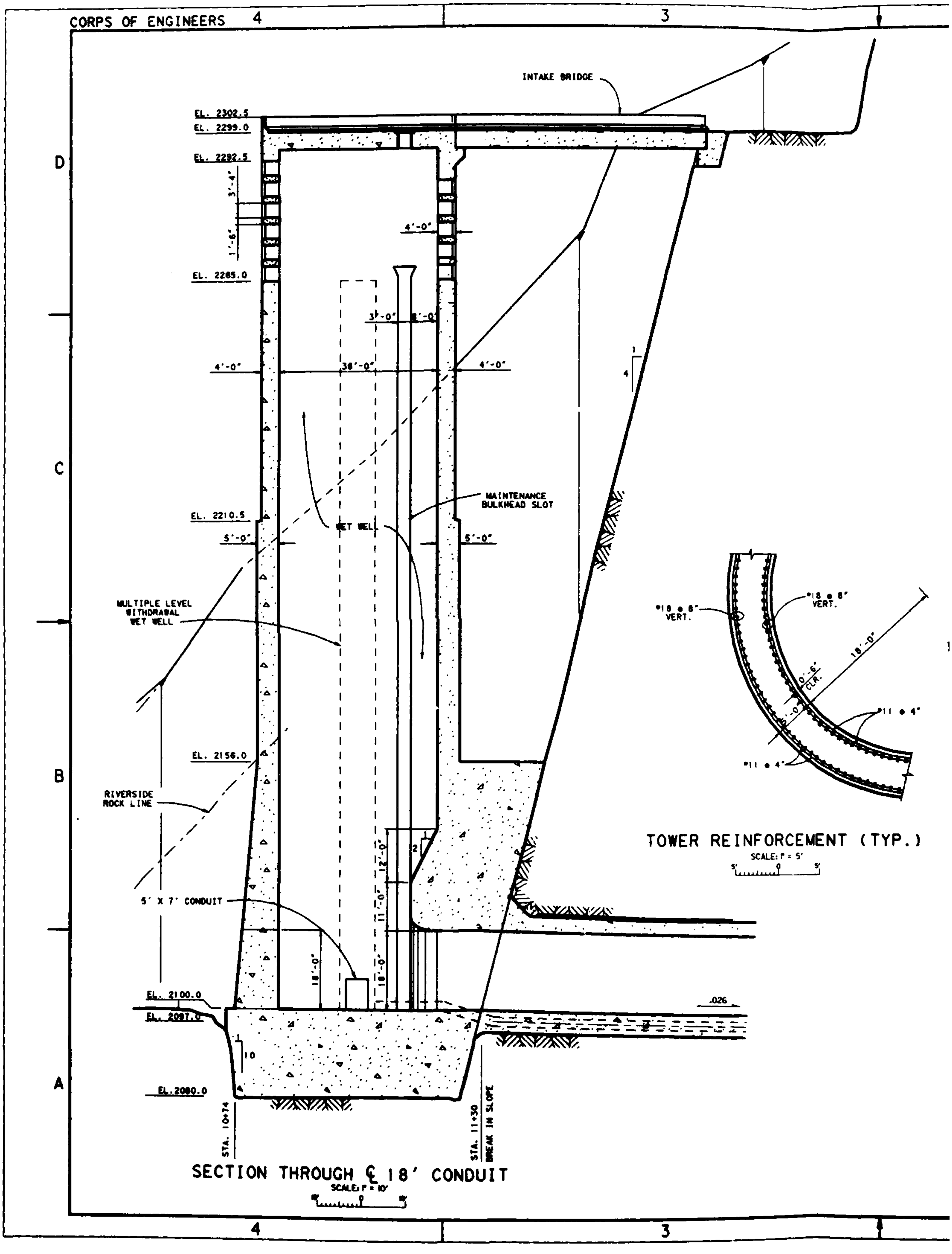
MID-TUNNEL CONTROL

DIVERSION/OUTLET TUNNEL
PLAN, PROFILE & SECTIONS

PLATE 4-2

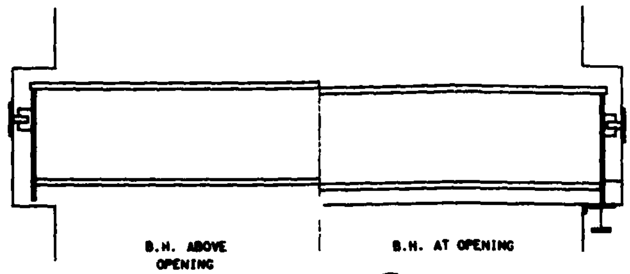
DISTRICT FILE NO. OAK-20-4/2

2 | 1



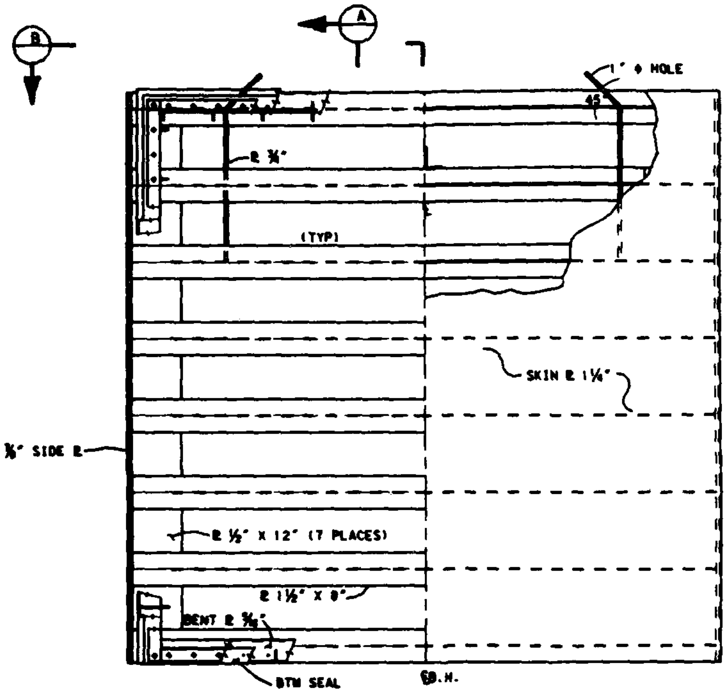
TOWER REINFORCEMENT (TYP.)
SCALE: 1" = 5'
SECTION THROUGH ℄ 18' CONDUIT
SCALE: 1" = 10'

U.S. ARMY

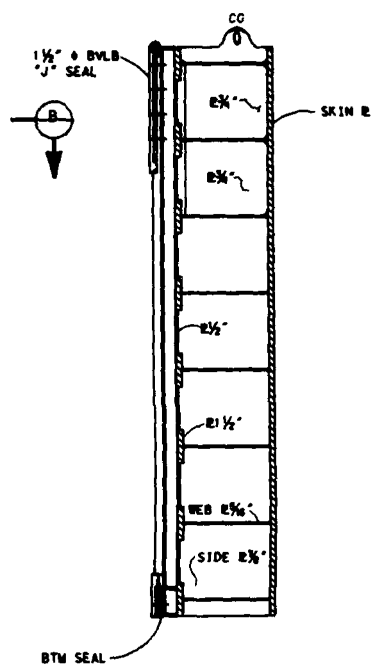


SECTION B

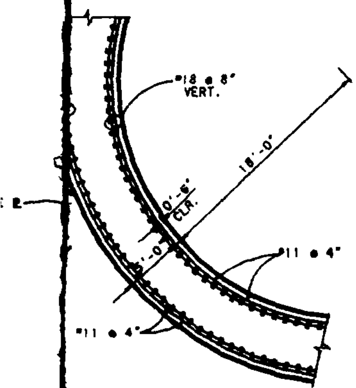
PLAN - R.O. BULKHEAD



ELEVATION
R.O. MAINTENANCE BULKHEAD



SECTION A



INFORCEMENT (TYP.)

SCALE: 1" = 5'

U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS	U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS

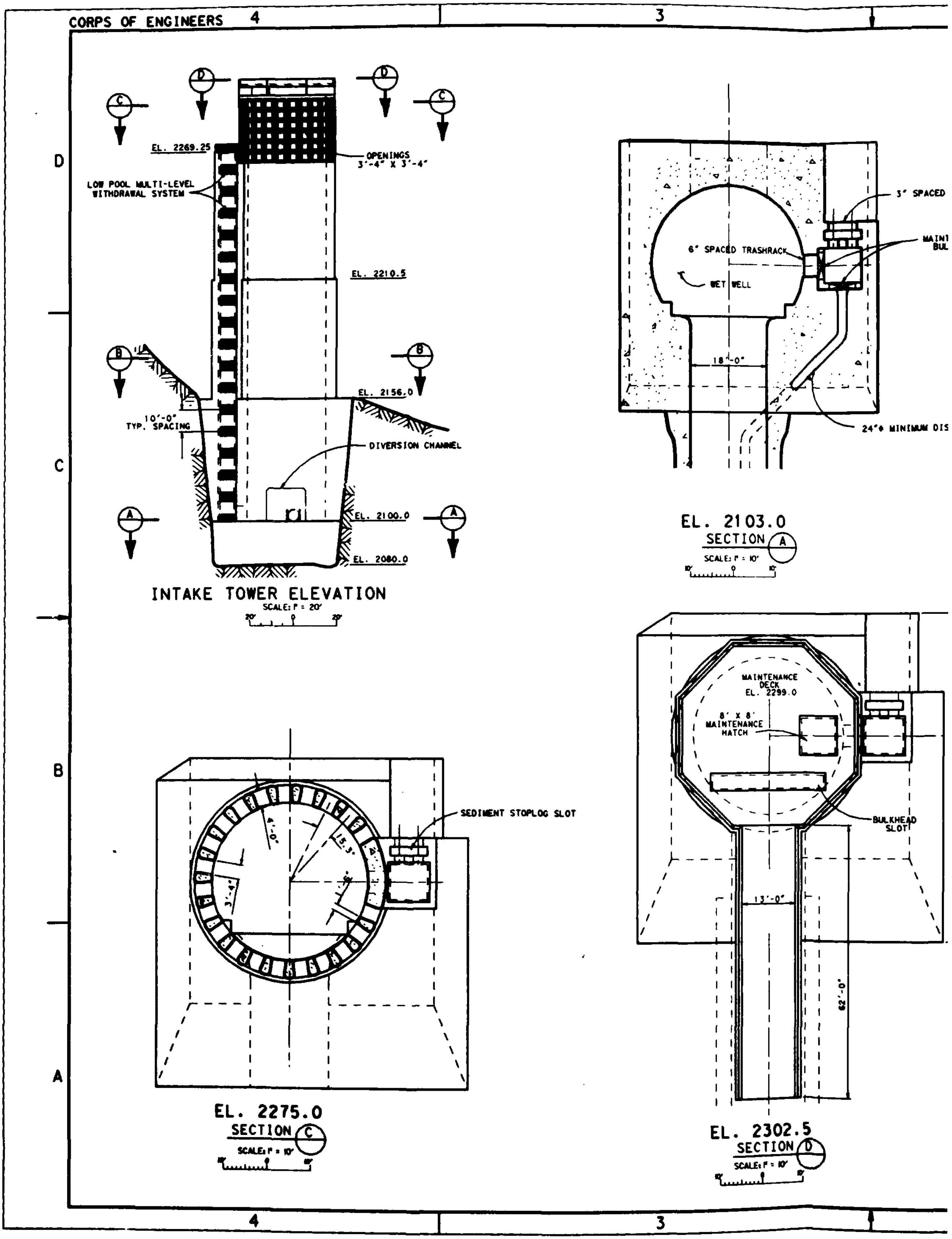
REVISIONS

DESIGNED BY: D. CHAMBERS
DRAWN BY: S. ERICSON
CHECKED BY: S. KONNO

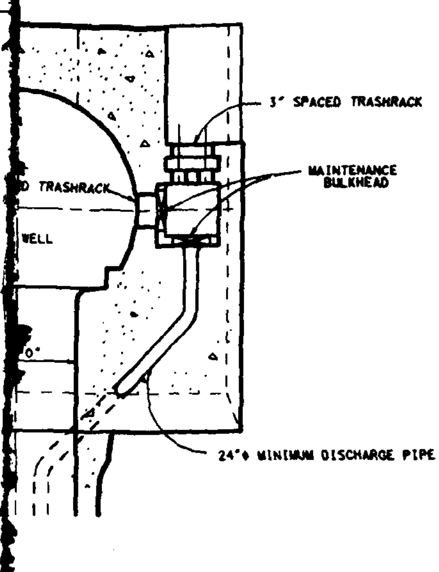
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
OUTLET WORKS
MID-TUNNEL CONTROL
INTAKE TOWER
SECTIONS & MISC. DETAILS

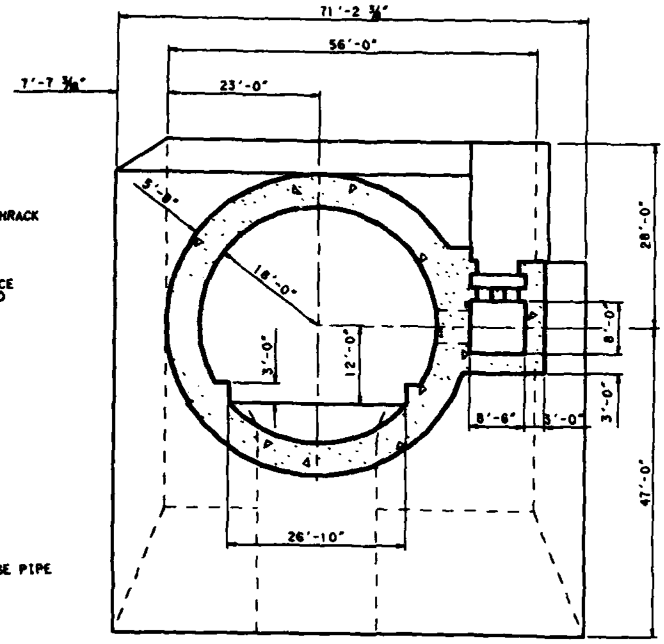
SUBMITTED BY: W. E. DAUGHERTY, P.E.
DATE APPROVED:
DISTRICT FILE NO. OAK-20-4/3
PLATE 4-3



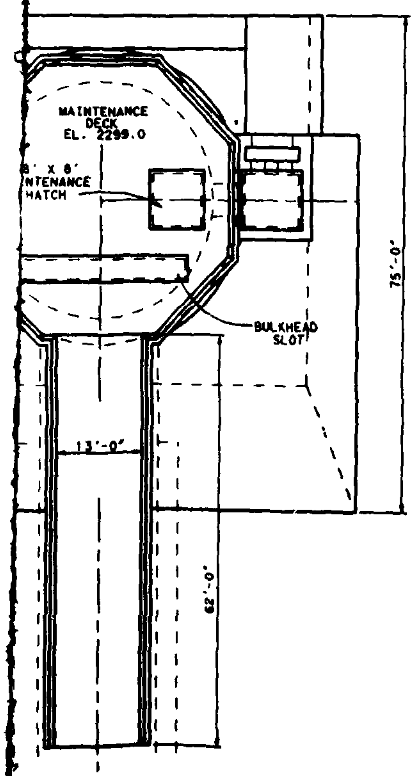
CORPS OF ENGINEERS
4
3
D
C
B
A
EL. 2269.25
OPENINGS 3'-4" X 3'-4"
LOW POOL MULTI-LEVEL WITHDRAWAL SYSTEM
EL. 2210.5
EL. 2156.0
10'-0" TYP. SPACING
DIVERSION CHANNEL
EL. 2100.0
EL. 2080.0
INTAKE TOWER ELEVATION
SCALE: 1" = 20'
3" SPACED
6" SPACED TRASHRACK
WET WELL
MAINT BUL
18'-0"
24"ɸ MINIMUM DIS
EL. 2103.0
SECTION A
SCALE: 1" = 10'
SEDIMENT STOPLOG SLOT
4'-0"
15.5°
3'-4"
EL. 2275.0
SECTION C
SCALE: 1" = 10'
MAINTENANCE DECK EL. 2299.0
8' X 8' MAINTENANCE HATCH
BULKHEAD SLOT
13'-0"
62'-0"
EL. 2302.5
SECTION D
SCALE: 1" = 10'



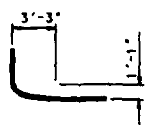
EL. 2103.0
SECTION A
SCALE: 1" = 10'



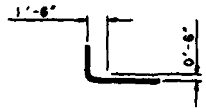
EL. 2175.0
SECTION B
SCALE: 1" = 10'



EL. 2302.5
SECTION D
SCALE: 1" = 10'



ROOF CURVE DATA
MINIMUM DISCHARGE CONDUIT
SCALE: 1" = 5'

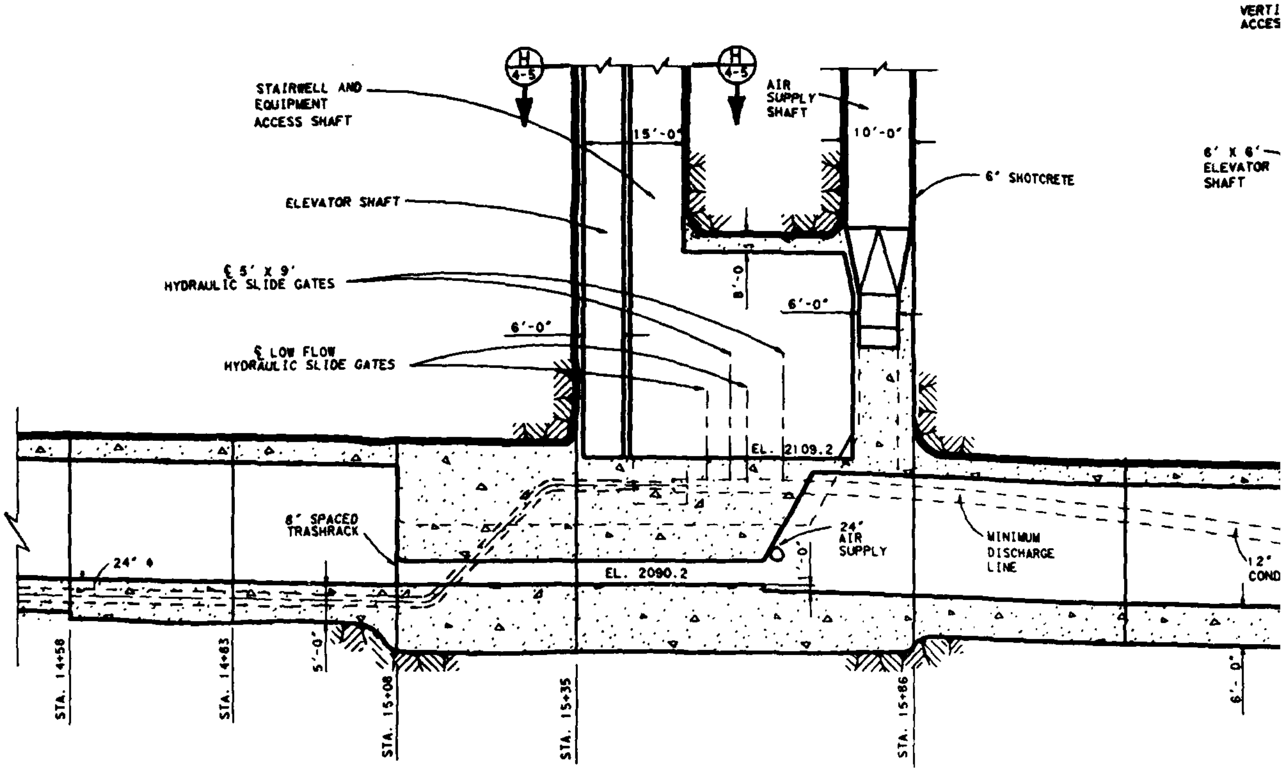


SIDE CURVE DATA
MINIMUM DISCHARGE CONDUIT
SCALE: 1" = 5'

U.S. ARMY

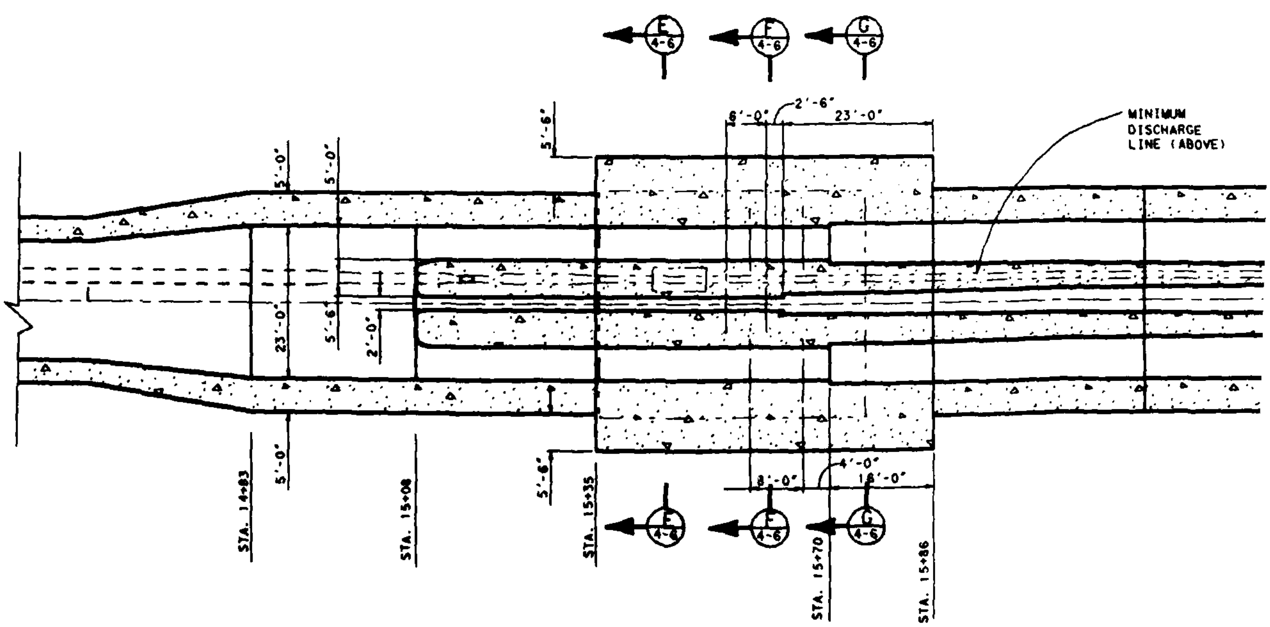
U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS	U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS
DESIGNED BY: D. CHAMBERS	SANTA ANA RIVER MAINSTEM, CALIFORNIA PHASE II GENERAL DESIGN MEMORANDUM
DRAWN BY: S. ERICSON	SEVEN OAKS DAM OUTLET WORKS
CHECKED BY: S. KONNO	MID-TUNNEL CONTROL INTAKE TOWER ELEVATION & PLAN SECTIONS
SUBMITTED BY: R. E. DAUGHERTY, P.E.	DISTRICT FILE NO. OAK-20-4/4
PLATE	4-4

CORPS OF ENGINEERS 4 3



PROFILE - MID-TUNNEL CONTROL

SCALE: 1" = 10'

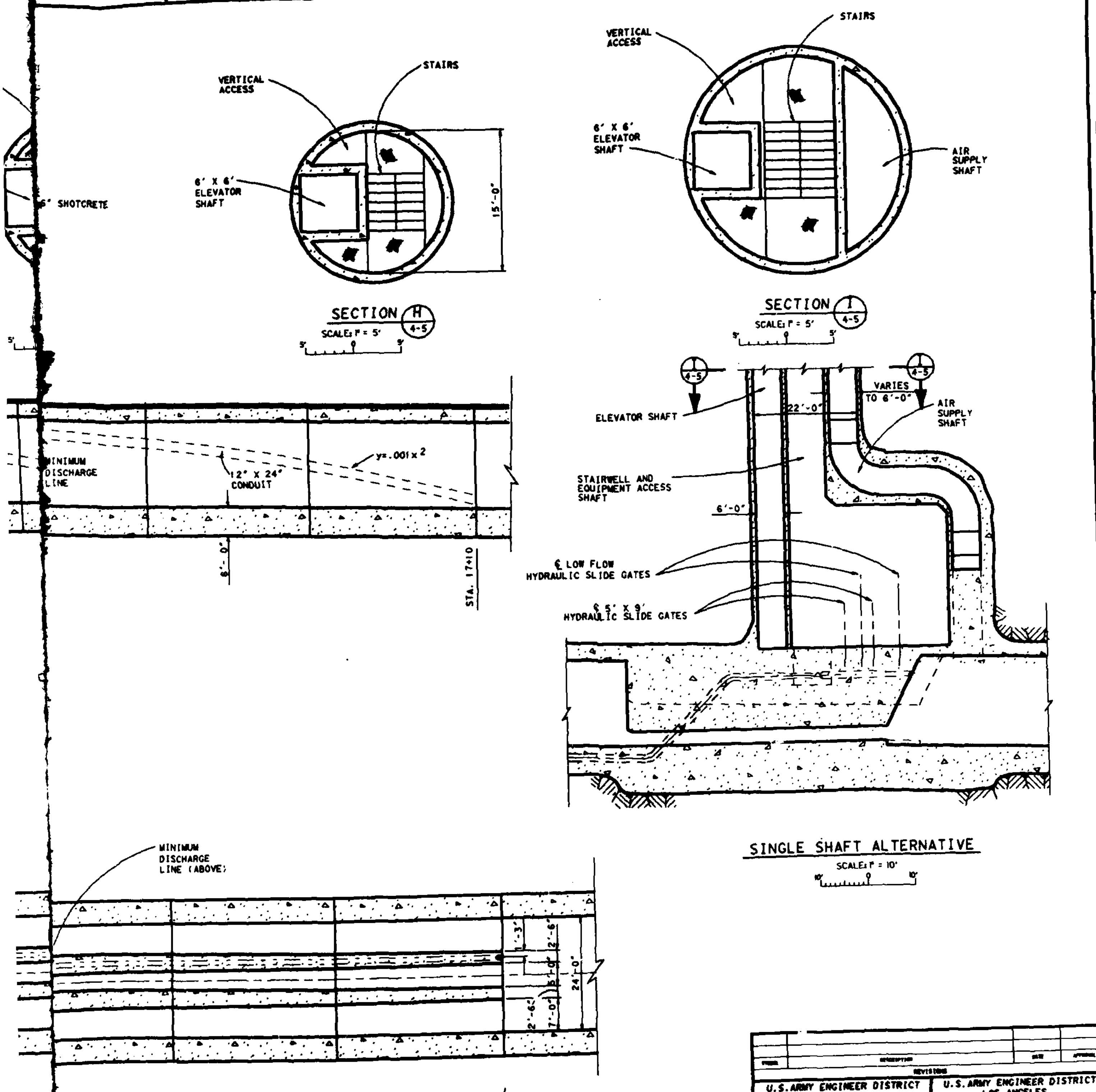


PLAN - MID-TUNNEL CONTROL

SCALE: 1" = 10'

4 3

U.S. ARMY



MARK	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS

U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS

DESIGNED BY: D. CHAMBERS
DRAWN BY: S. ERICSON
CHECKED BY: K. SWANSON

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
OUTLET WORKS
MID-TUNNEL CONTROL
CONTROL
PLAN AND PROFILE

SUBMITTED BY: W. E. DAUGHERTY, P.E. CHIEF, DESIGN BRANCH

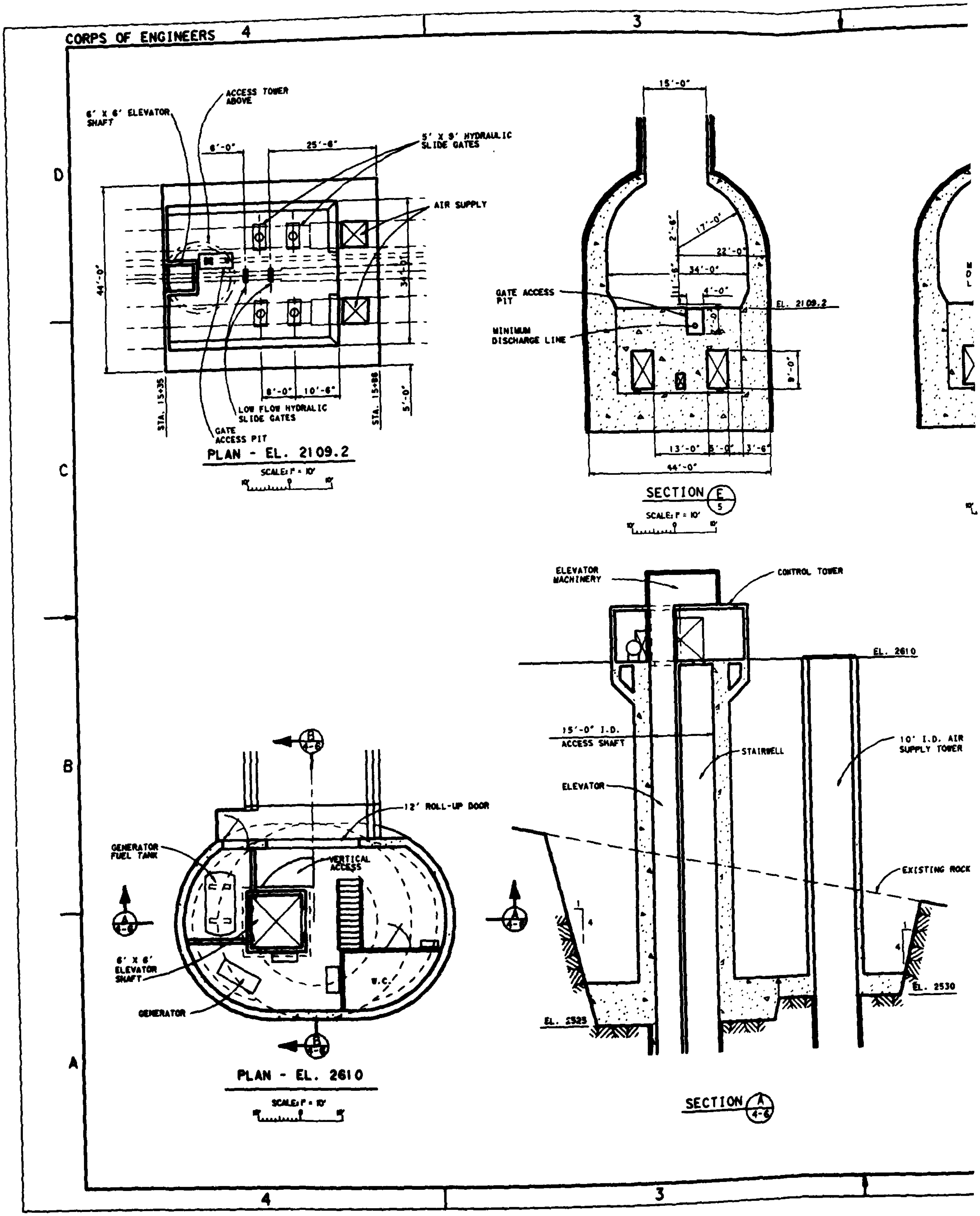
DATE APPROVED:

SPEC. NO. DACW09- -

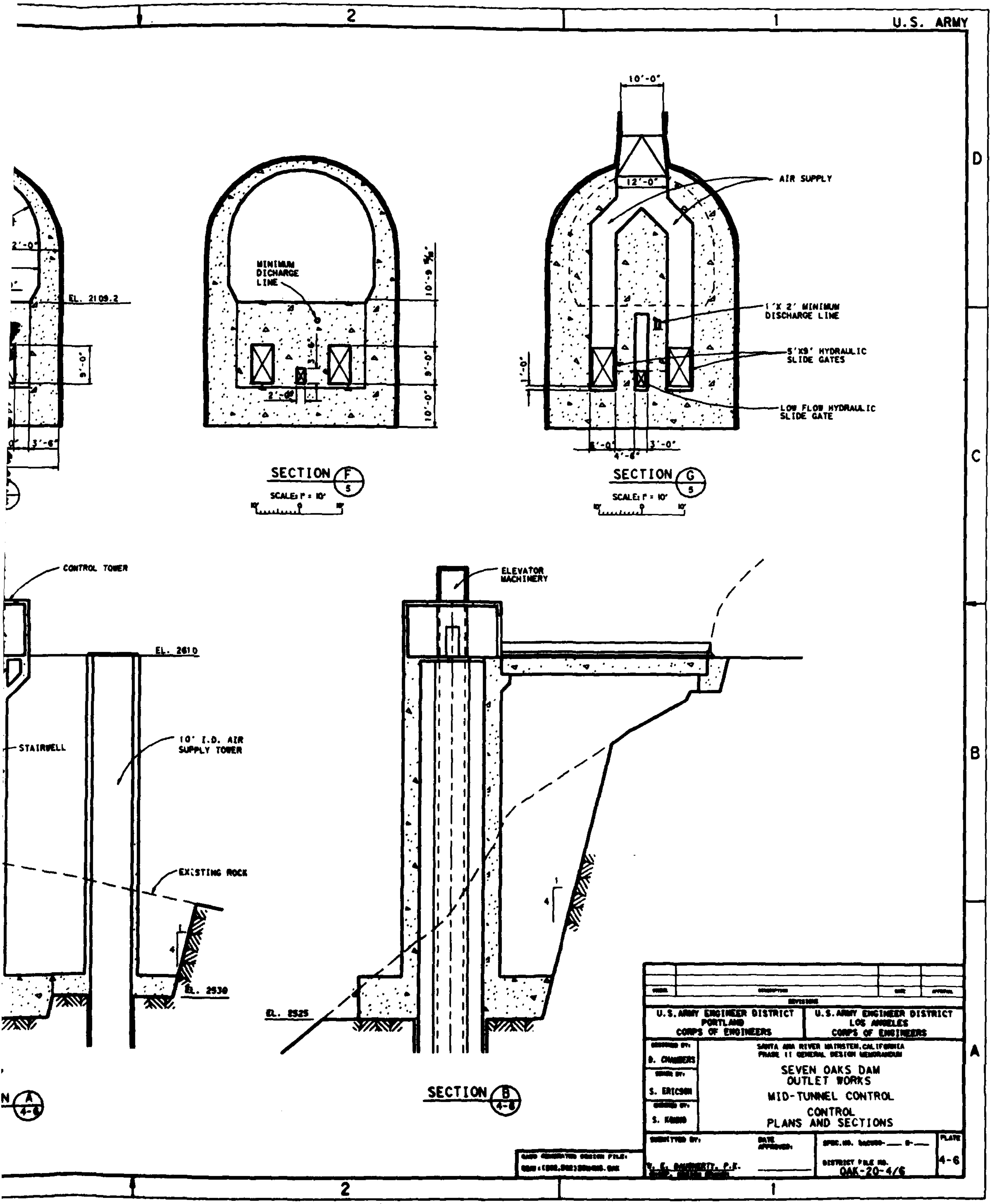
DISTRICT FILE NO. OAK-20-4/5

PLATE 4-5

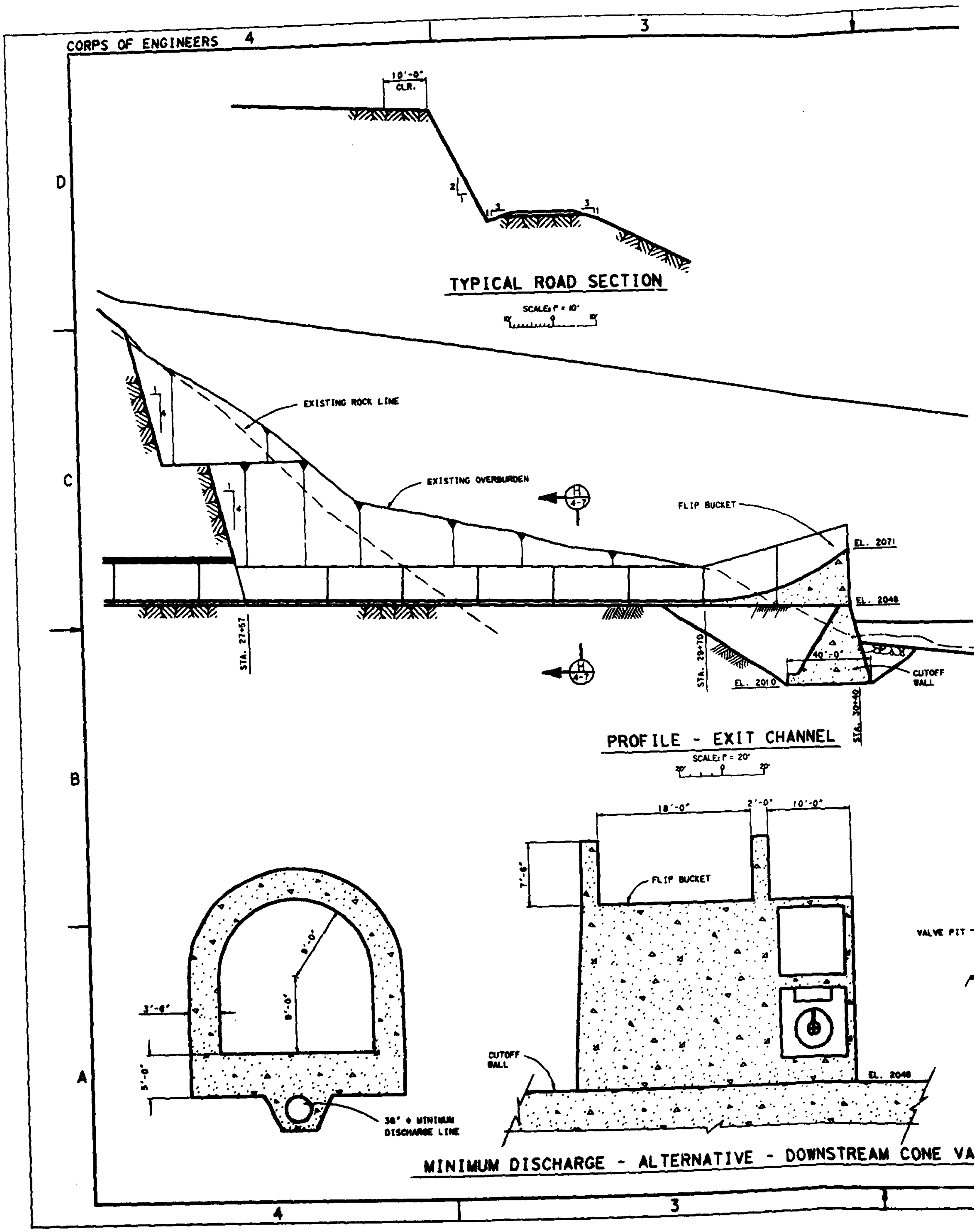
CADD GENERATED DESIGN FILE: DGN:[200,200]20X4X5.OAK



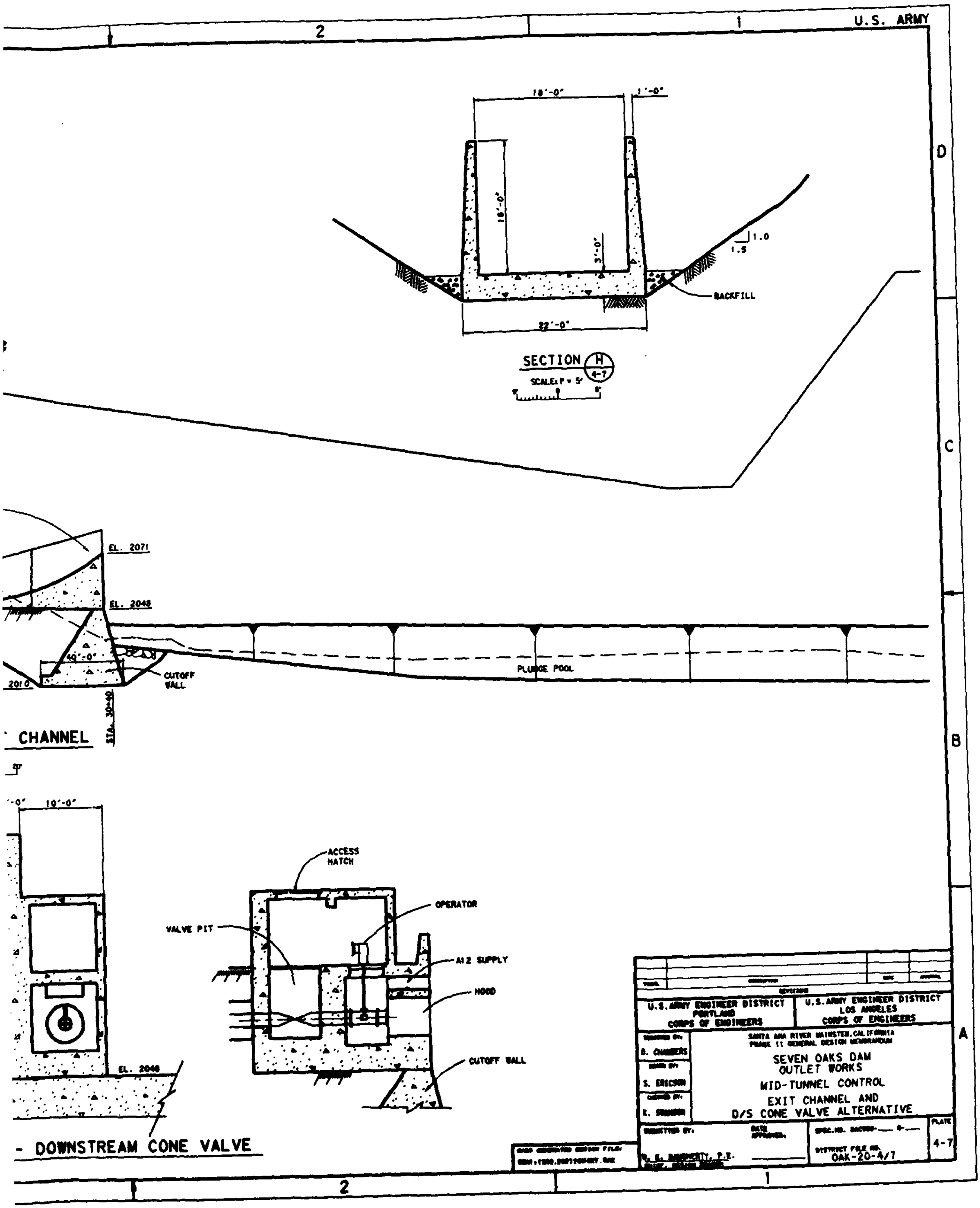
CORPS OF ENGINEERS
4
3
D
C
B
A
ACCESS TOWER ABOVE
6' X 6' ELEVATOR SHAFT
6'-0"
25'-6"
5' X 9' HYDRAULIC SLIDE GATES
AIR SUPPLY
44'-0"
34'-0"
8'-0"
10'-6"
5'-0"
STA. 15+35
STA. 15+86
LOW FLOW HYDRAULIC SLIDE GATES
GATE ACCESS PIT
PLAN - EL. 2109.2
SCALE: 1" = 10'
15'-0"
17'-0"
22'-0"
34'-0"
4'-0"
2'-6"
GATE ACCESS PIT
MINIMUM DISCHARGE LINE
EL. 2109.2
9'-0"
13'-0"
3'-0"
3'-6"
44'-0"
SECTION E/5
SCALE: 1" = 10'
ELEVATOR MACHINERY
CONTROL TOWER
EL. 2610
15'-0" I.D. ACCESS SHAFT
STAIRWELL
10' I.D. AIR SUPPLY TOWER
ELEVATOR
EXISTING ROCK
EL. 2525
EL. 2530
SECTION A/4-6
12' ROLL-UP DOOR
GENERATOR FUEL TANK
VERTICAL ACCESS
6' X 6' ELEVATOR SHAFT
GENERATOR
W.C.
PLAN - EL. 2610
SCALE: 1" = 10'



U.S. ARMY
10'-0"
12'-0"
AIR SUPPLY
MINIMUM DICHARGE LINE
1'X 2' MINIMUM DISCHARGE LINE
5'X9' HYDRAULIC SLIDE GATES
LOW FLOW HYDRAULIC SLIDE GATE
EL. 2109.2
10'-9 5/8"
9'-0"
10'-0"
2'-0"
3'-6"
3'-0"
4'-6"
SECTION F 5
SCALE: 1" = 10'
SECTION G 5
SCALE: 1" = 10'
CONTROL TOWER
ELEVATOR MACHINERY
EL. 2610
STAIRWELL
10' I.D. AIR SUPPLY TOWER
EXISTING ROCK
EL. 2530
EL. 2525
SECTION A 4-6
SECTION B 4-6
REVISIONS
U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS
U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS
D. CHAMBERS
S. ERICSON
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
OUTLET WORKS
MID-TUNNEL CONTROL
CONTROL
PLANS AND SECTIONS
SUBMITTED BY:
DATE APPROVED:
DISTRICT FILE NO.
OAK-20-4/6
PLATE
4-6



CORPS OF ENGINEERS
4
3
D
C
B
A
10'-0"
CLR.
TYPICAL ROAD SECTION
SCALE: 1" = 10'
EXISTING ROCK LINE
EXISTING OVERBURDEN
FLIP BUCKET
EL. 2071
EL. 2048
EL. 2010
40'-0"
CUTOFF WALL
STA. 27+57
STA. 29+70
STA. 30+40
H
4-7
PROFILE - EXIT CHANNEL
SCALE: 1" = 20'
18'-0"
2'-0"
10'-0"
7'-6"
FLIP BUCKET
VALVE PIT
CUTOFF WALL
EL. 2048
3'-0"
9'-0"
5'-0"
36" ϕ MINIMUM DISCHARGE LINE
MINIMUM DISCHARGE - ALTERNATIVE - DOWNSTREAM CONE VA



U.S. ARMY
18'-0"
1'-0"
16'-0"
3'-0"
1.0
1.5
BACKFILL
22'-0"
SECTION H 4-7
EL. 2071
EL. 2048
40'-0"
CUTOFF WALL
PLUNGE POOL
STA. 30+40
CHANNEL
10'-0"
ACCESS HATCH
OPERATOR
VALVE PIT
AIR SUPPLY
HOOD
CUTOFF WALL
EL. 2048
- DOWNSTREAM CONE VALVE
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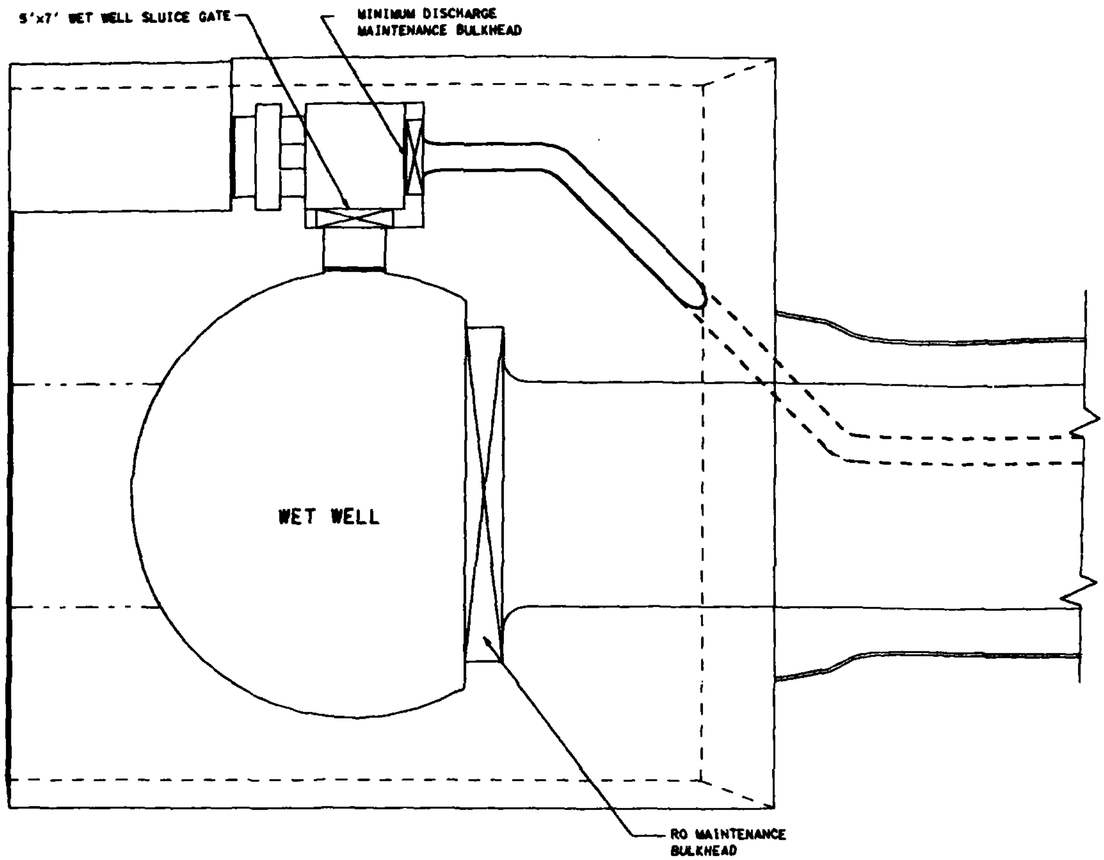
CORPS OF ENGINEERS
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM
D. CHAMBERS
S. ERICSON
SEVEN OAKS DAM
OUTLET WORKS
MID-TUNNEL CONTROL
EXIT CHANNEL AND
D/S CONE VALVE ALTERNATIVE
PLATE
4-7
DISTRICT FILE NO.
OAK-20-4/7

CORPS OF ENGINEERS 4 3

D

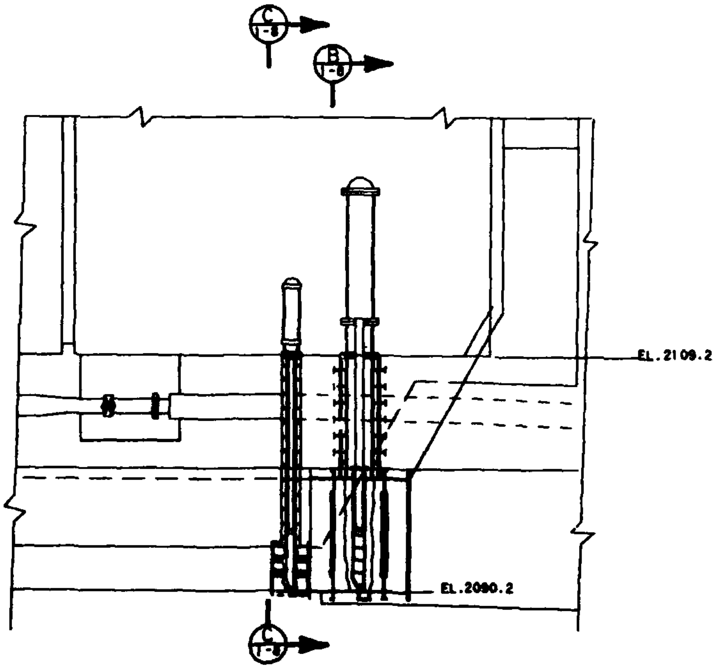
C



PLAN - INTAKE TOWER

SCALE: " = 1'-0"

B



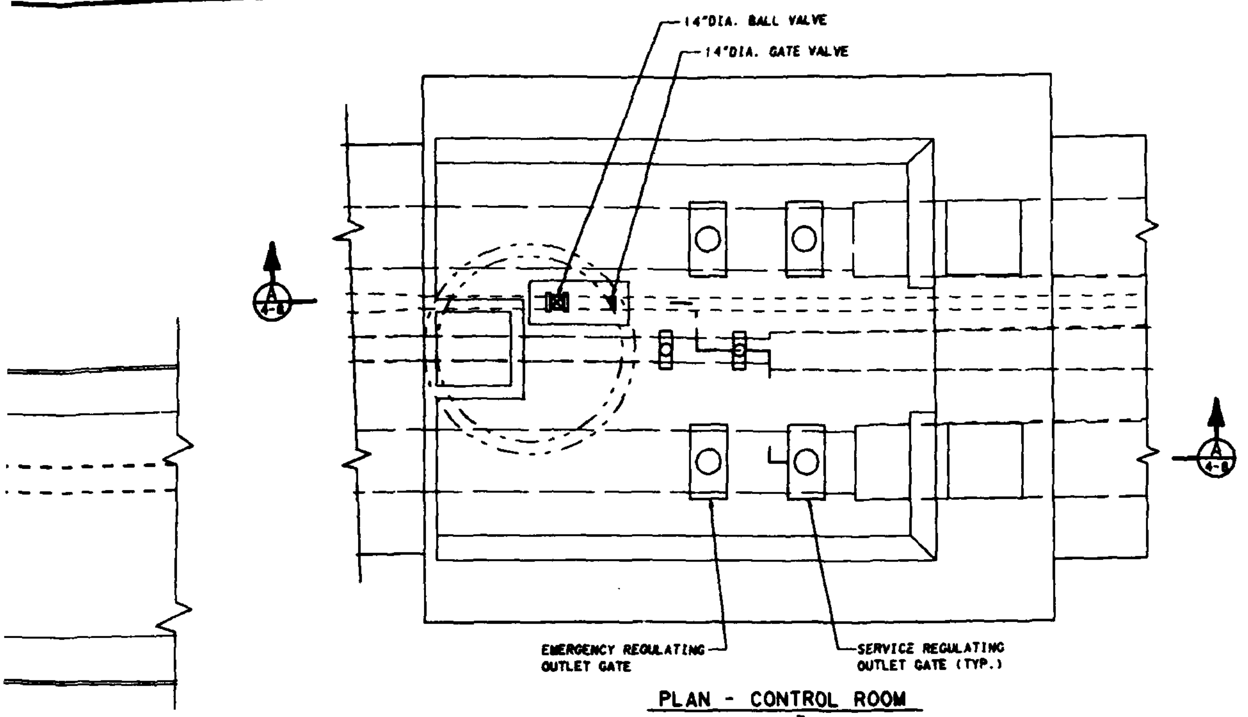
A

SECTION A 4-8

SCALE: " = 1'-0"

4 3

U.S. ARMY



PLAN - CONTROL ROOM

SCALE: 3/8" = 1'-0"

NOTES:
1. SEE DRAWING OAK-20-1/8 FOR FRONT ELEVATION OF SLIDE GATE AND HYDRAULIC SCHEMATIC DIAGRAM.

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SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
OUTLET WORKS
MID-TUNNEL CONTROL
MECHANICAL EQUIPTMENT LAYOUT

DESIGNED BY: R.M.
DRAWN BY: D.B.
CHECKED BY: R.M.

SUBMITTED BY: V. E. DAUGHERTY, P.E.
DATE APPROVED:

SPEC. NO. DACW09-__ B-__
DISTRICT FILE NO. OAK-20-4/8
PLATE 4-8

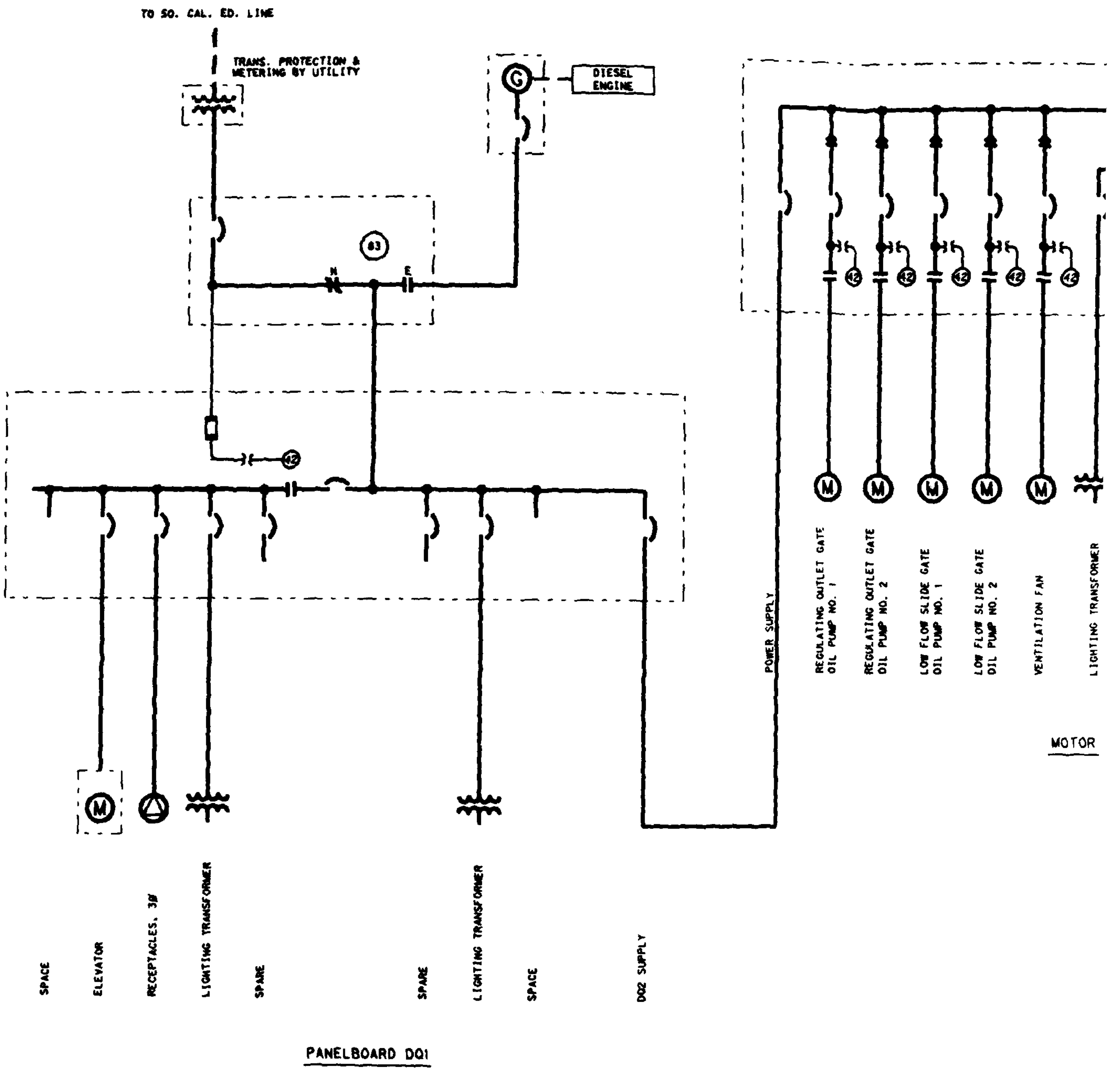
CORPS OF ENGINEERS 4 3

D

C

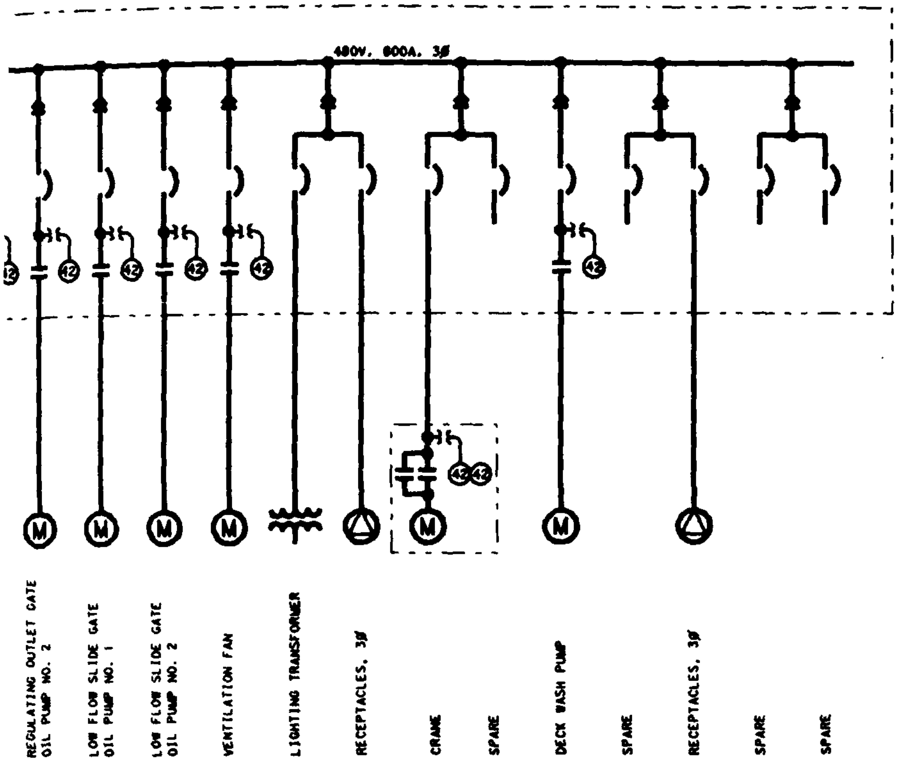
B

A



4 3

U.S. ARMY



MOTOR CONTROL CENTER DQ2

LEGEND

G GENERATOR

M MOTOR

RECEPTACLE, 3Ø, 480V

CIRCUIT BREAKER OR MOTOR CIRCUIT PROTECTOR

42 MOTOR STARTER OR CONTACTOR

83 AUTO-TRANSFER SWITCH
E-EMERGENCY
N-NORMAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT
PORTLAND
CORPS OF ENGINEERS

U.S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY: RTD
DRAWN BY: JLH
CHECKED BY: KAB

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
OUTLET WORKS
MID-TUNNEL CONTROL
ONE-LINE DIAGRAM

SUBMITTED BY:
DATE APPROVED:
W. E. DAUGHERTY, P.E.

SPEC. NO. DACW09-___ B-___
DISTRICT FILE NO. OAK-20-4/9

PLATE 4-9

SECTION 5

COMPARISON OF THE OUTLET WORKS ALTERNATIVES

5.1 General. Evaluation of the Seven Oaks Outlet Works is separated into three areas (criteria): earthquake survivability, outlet works costs, and operability (O&M). The final design selection is based primarily on the best alternative with regards to its ability to "survive" the design earthquake and associated consequences and construction costs.

5.2 Advantages and Disadvantages. A broad listing of advantages and disadvantages are summarized for the three alternatives as follows:

a. Upstream Control.

Advantages	Disadvantages
Non-pressurized tunnel	Maximizes tunnel excavation
Minimizes exit velocity	Maximizes liner construction
If tunnel shears, can minimize tunnel erosion damage with control at upstream	Maximizes tower construction
	Two bench tunnel construction required
	Risk of losing gate access under tunnel displacement scenario
	Safe access to gates may be delayed, following an earthquake, due to aftershocks
	No repair bypass
	Tunnel/channel cavitation potential

	Large air demand
	Difficult to drain pool if earthquake jams gates at small openings
	Maintenance of minimum discharge line
	Cavitation potential - minimum discharge line

b. Mid-Tunnel Control.

Advantages	Disadvantages
Gate chamber access improved over upstream control	24-inch conduit maintenance
	No bypass downstream of gates
Dual shaft design is amiable to economies of raise bore mining techniques	No high pool control upstream of gates
	Potential for cavitation in minimum discharge line
	Requires upstream cofferdam with two heading tunnel construction
	Low flow trashrack cleaning
	Cavitation potential in tunnel
	Maximizes construction efforts features, and sequencing
	Greatest change order potential

Shaft/tunnel intersection complex and expensive to build

High damage potential due to shear displacement in shaft/tunnel and controls

Safe access to gates may be delayed, following an earthquake, due to aftershocks

Shaft siting, analysis, and explorations extensive

c. Downstream Control.

Advantages

Separate exit chutes

Operational flexibility

Low cavitation potential

Simple aeration

Minimizes tunnel construction

Good access to controls

Overbuild for seismic displacement

Minimizes use of 11-foot RO

Built-in diversion capability

Future power potential

Best gate/control earthquake survivability

Disadvantages

Maintenance of conduits

High velocities in 11-foot RO conduit

High exit velocity (160 fps)

Potential for dynamic water loads on RO conduit

Complex and expensive upstream emergency gate if required

Seismic displacement may pressurize tunnel downstream of embankment

Complex diversion sequence

5.3 Evaluation Criteria. The principal criteria which influence alternative selection the greatest are earthquake survivability and construction costs. Earthquake survivability, functionality, and dam safety were all considered to encompass dam safety concerns and are assumed to have the same objectives: reservoir control, access, inspection, and repair. At the January 1988 TRC these objectives were presented as the ability to control the reservoir after a major earthquake; the capability to inspect the outlet facilities following a major earthquake; and access for repair following a major earthquake. In a CESPD-ED-PC letter dated 14 March 1988, Subject: "Seven Oaks Outlet Works - Design Critera," CESPD directed the use of the following criteria at the urging of the USACE Dam Safety Office. Specifically, "Design of the outlet works would have to account for design earthquake requirement of 0.7g rock acceleration and up to 4 feet displacement in any direction on any one of many planes," and "the design would have to demonstrate the best probability to provide positive control of the reservoir should this event occur." To further qualify the displacement, if the postulated maximum 4-foot displacement should occur on a single plane, that plane would be a significant shear already in existence and should be identifiable. Any displacement, however, would most likely be distributed unequally among these and lesser shears. Therefore, the displacement literally could occur along any of these planes of weakness and in any amount up to a cumulative total not to exceed 4 feet through the site. The criteria requires that it be possible to either store or release any pool behind the dam at the discretion of the operator following the design earthquake. As presented, it is felt that this criteria requires further discussion and clarification. With respect to the Seven Oaks project, the first priority following a major earthquake will be to draw down the reservoir. Due to the threats of aftershocks, no access or inspection will be allowed until the reservoir is lowered and the upstream maintenance bulkhead is in-place. The first criteria is then to allow reservoir withdrawal immediately following a major earthquake. It would be undesirable to utilize an upstream emergency gate under high pool with the threat of aftershocks. A jammed upstream gate in a lowered position, under a high pool, would be unaccessible for repair.

The outlet alternatives are evaluated by comparing the key system components and their respective capabilities to survive earthquake deformation and shaking. Key components are such features as gate systems, control structures, tunnel, conduits, accesses, bridges, towers, and concrete plugs. These features are evaluated, each in turn with respect to their impact on survivability and dam safety. Construction cost is another principal cost item to be evaluated. Other items considered, but of a secondary nature relative to selection are:

Cost items:	*Operations and maintenance*
Constructibility:	Diversion Tunnel/shaft/excavation Schedule
Operability:	System reliability - gates under high pools Gate maintenance Conduit/tunnel maintenance Tower maintenance activities

5.4 Evaluation of Principal Criteria. Earthquake survivability and construction costs are evaluated for each of the three alternatives.

a. Earthquake Survivability. Each key component of the outlet works is evaluated with respect to the occurrence of the design earthquake with full pool and the resulting impact to meeting the survivability objectives; i.e., maintaining reservoir control, inspection capability, etc. The project's response to the design earthquake with a dry reservoir (or debris pool only) is not considered pertinent to the alternative selection. The joint occurrence of the design earthquake and a flood has an associated risk which can also be considerea in the final evaluation. Risk values for the joint occurrence of the earthquakes and flood are shown in table 5-1.

Table 5-1. Risk Value for Joint Earthquake and Flood at Seven Oaks Dam Site*

Flood Return Period (pool elevation)	Earthquake Return Period		
	10 Years	100 Years	1,000 Years
10 yr (2,400)	1.9 x E-2	1.9 x E-3	1.9 x E-4
100 yr (2,530)	1.9 x E-3	1.9 x E-4	1.9 x E-5
1,000 yr (2,592)	1.9 x E-4	1.9 x E-5	1.9 x E-6

*Reference Hynes-Griffin, Mary Ellen, The Joint Occurrence of Earthquake and Floods, Misc. Paper GL-80-10 WES, September 1980. The values assume a 100-year project life and a 1-week flood duration.

Cumulated flood days at specific elevations over the 100-year project life are depicted on table 5-5.

(1) Positive Control (Gate Chamber Survivability) with Full Pool and Design Earthquake. The best probability to provide positive control of the reservoir in the event of the design earthquake is evaluated.

(a) Upstream Control. With the upstream control alternative, all the gates will be in a single monolith located within the rock mass at the upstream portal. Should displacement occur at the portal, the monolith may shear in conjunction with the rock due to the monolith embedment and the forces required to move this rock. Rock confinement and nearness to the surface, however, may allow the local rock stresses to relieve themselves leaving the monolith intact. The gates may or may not displace relative to the rest of the gate monolith. There is an additional chance that the gate chamber may separate from the tunnel or from the tower, and that significant leakage could occur resulting in some loss of positive control. Separation of the gate chamber at the downstream end would most likely result in flooding the gate chamber, rendering the electrical equipment inoperable even though power and control cables will likely be intact. Positive control cannot be guaranteed with this alternative.

(b) Mid-Tunnel Control. With the mid-tunnel control alternative, all of the gates will be in a single location, housed in a

chamber in the rock mass some 400 feet downstream of the tower. This location is most favorable from the standpoint of potential damage due to ground shaking. Should rock displacement (even if only a few inches) occur at the gate location, the walls of the chamber will echo this shear, as will the gate components within the structure, resulting in significant damage and loss of positive control. A rupture of the pressurized portion of the gate chamber would flood the access shaft, making access to the gates impossible. Displacement of the access shaft may damage the stairs, elevator, ductwork, and concrete lining, could also sever access, control to the gate chamber, or choke the bottom of the shaft with fallen debris. A rock shear may also surface in the reservoir, causing a flooding of the shaft and gate room from inflow from the pool above. Electrical equipment may become inoperable and power and control cables will likely not survive. Positive control cannot be guaranteed with this alternative.

(c) Downstream Control. There are two options to this alternative:

(c.1) With the first option, all of the gates will be in a single monolith located outside the rock mass at the downstream tunnel portal. Should a displacement occur at the chamber, it will most likely move as a monolith, i.e., there will be no relative displacement of the gates with respect to the rest of the chamber. The mechanical equipment of the gates, therefore, stand a good chance of surviving the displacement. Under this scenario the monolith would likely separate from the outlet conduit, and significant leakage could occur upstream of the gates resulting in some loss of positive control. Much of the leakage may be controlled if the concrete plug connecting the gate chamber and the tunnel remains intact. The extent of damage and leakage will depend on the location and nature of the displacement (horizontal or vertical movement). Access to this gate location is easy, and power and control circuits should remain intact. This alternative has the best chance of providing positive control for any of the single gate chamber alternatives, under the fault displacement scenario.

(c.2) The second downstream control option includes the addition of a gate chamber upstream to be used in the event that

downstream damage and resulting leakage is unacceptable under a high pool scenario. The redundancy of providing the second gate location assumes that significant displacement will not occur at both locations during the same earthquake event. The single upstream gate will have a chamber similar to that for upstream control. Due to the larger size and full pool loading, this gate may be a hydraulic operated roller type in lieu of a hydraulic slide gate. The downstream chamber will be identical to the first option with the same advantages and disadvantages.

There is a cost/performance trade-off involved in providing the second gate chamber. Providing the upstream emergency gate may well add 3 to 5 million dollars to the cost of the alternative. The upstream gate chamber would have the same survivability as the upstream control gate chamber and access is limited to low pools. With a high pool, inspection of the upstream gate system would be impossible. Under such an event, a blind attempt to close the upstream gate would have risks associated with it. If the gate jammed partially open, it would not provide the relief needed to repair the main gates, and it may be impossible to drain the reservoir. Thus, it is possible to go from a situation where there isn't positive control, but there is ability to drain the reservoir safely, to a situation where neither is available.

(2) Gate Failure due to Shaking. This criterion was orally proposed by the USACE Dam Safety Office in January 1988. This criterion requires that the magnitude of shaking (due to the design earthquake) that would occur at the gates be evaluated for each outlet works alternative, and an assessment made of the likelihood of the gates being rendered inoperative by this shaking.

(a) Upstream Control. With the upstream control alternative the gates are likely to experience the full 0.7g ground motion acceleration should the design earthquake occur. No cases of record have been found which document a failure of this gate type due to shaking. The gate components will be designed for the dynamic stresses using traditional analysis methods. If this failure scenario is still of concern, model testing could be conducted to qualify design stresses.

(b) Mid-Tunnel Control. With the gates embedded in the rock mass, the gates are likely to experience less than the full 0.7g ground motion acceleration expected at the surface should the design earthquake occur. As explained above, properly designed gates would function after experiencing the design earthquake.

(c) Downstream Control. With the downstream control alternative the gates are likely to experience the full shaking of the design earthquake. As explained above, properly designed gates should survive this.

(3) Gate Failure Due to Displacement. This criterion was adopted early in the design process. It requires that the magnitude of the local shear displacement at the gate location be evaluated for each outlet works alternative in the case that a local shear displacement of up to 4 feet should occur due to an earthquake. In addition, an assessment would be made of the likelihood of the gates being rendered inoperative by the local displacement.

(a) Upstream Control. With the upstream control alternative, all of the gates will be housed in a chamber located in the upstream portal rock mass. The chamber is restrained on five sides by the rock mass, see plate 2-3. Should a displacement occur at the chamber it will most likely echo through the structure and into the gate components. Due to the restraint, the rock shearing forces will pass through the structure instead of being redistributed or dissipated. The gate chamber and gate components cannot be designed for forces of this magnitude, thus resulting in a high probability of damage and failure of the gating system.

(b) Mid-Tunnel Control. With the mid-tunnel control, all of the gates will be located in a single chamber deep within the rock mass. Shearing is expected to be greater than that indicated for the upstream alternative; likewise, probability is high for chamber and gate damage rendering the system inoperable.

(c) Downstream Control. With downstream control, all of the gates will be in a single location outside of the rock mass at the downstream end. Should a displacement occur at the chamber, it will most likely move as a monolith, i.e., there will be no relative displacement of the gates with the rest of the chamber. The mechanical equipment and structure of the gates, therefore, stand a better chance of surviving a displacement.

(4) Conduit/Tunnel Failure. This criterion requires that the potential for conduit (and/or tunnel) failure as a result of the design earthquake, or attempting to make reservoir releases through an earthquake damaged conduit, be evaluated for each alternative. The impact of aftershocks, which may cause additional damage several months after the main event, is also a concern. In general, tunnels have an excellent record of performance under shaking and deformation. For seismic shaking with peak accelerations (at the surface) greater than about 0.5g, however, moderate to severe tunnel damage should be anticipated based on historical data. Complete collapse and loss of functionality would not be expected, though. Historically, lightly reinforced tunnel liners have performed better in areas of high seismicity due to greater liner flexibility. Regardless of tunnel shape, it is expected that a fault displacement(s) would shear the walls equally with any of the alternatives. Defensive measures may be incorporated at significant known shear features. A major collapse during strong shaking or fault displacement is not probable.

(a) Upstream Control. With this alternative the conduit consists mainly of an oblong-shaped cross section, with an inside width and height of 18 and 32 feet, respectively. Of all the alternatives this one has the largest overall conduit, and as such is the one most likely to sustain damage should the design earthquake occur. Additional damage could also occur during aftershocks. At each location that there is a displacement of the conduit, it is likely that cavitation and water induced erosion will occur. Damage may cease after the conduit discontinuity is eliminated. However, experience at other projects has demonstrated that once cavitation damage has been initiated, the amount of damage can increase rapidly until flow velocities are decreased. If the

service gate (or emergency gate) is still operational, flows through the conduit could be limited, which would reduce the amount of flow-induced damage. This would mean that it would take longer to drain the reservoir. If the embankment fails when there is a high pool, it would lead to a flood wave causing catastrophic downstream flood damage. It is felt that draining the reservoir as soon as possible after a major earthquake should be a high priority.

(b) Mid-Tunnel Control. With this alternative most of the tunnel section consists of a horseshoe-shaped tunnel with an inside diameter of 18 feet. This cross section is smaller than the upstream control alternative and, therefore, is less likely to be subject to damage. In the transition zones upstream and downstream of the gate chamber there are clear span sections 18 feet high and 24 feet wide. Because of the larger span, the tunnel walls are the thickest and excavation width and height are maximized. The larger tunnel size of these zones may contribute to the likelihood of sustaining damage during the design earthquake and potential strong aftershocks. Displacements that occur upstream of the control structure are not likely to initiate cavitation because of the high pressures and the low flow velocities (less than 30 fps) in the conduit. It is also unlikely that significant erosion, due to flow, will occur. Displacements that occur downstream of the control structure will have the same impact as described for the upstream control alternative. Again, draining the reservoir as soon as possible after a major earthquake should be a high priority.

(c) Downstream Control. With this tunnel, most of the section consists of a horseshoe shape with an inside diameter of 18 feet. The section has thin walls and is lightly reinforced relative to the other alternatives. As such, this tunnel, because of greater flexibility, may be less likely to sustain damage during the design earthquake. The actual outlet conduit is a 11-foot-diameter steel conduit located within the diversion tunnel. In the event of a 4-foot displacement in any direction, significant buckling and even rupturing of the steel conduit will take place. The steel conduit would have to be rigidly secured to the tunnel side walls and/or floor to prevent the high velocity re-entrant flow from

creating progressive cavitation damage to the conduit. At worst, if the conduit isn't rigidly secured, total tunnel blockage would be possible; however, a catastrophic failure would not be realized. If the conduit ruptures, the discharge for some distance downstream of the break will be at atmospheric pressure until the tunnel fills with water and flow is pressurized. There may be significant damage to a short reach of the tunnel, downstream of the break in the conduit, from flow-induced erosion and cavitation until the tunnel becomes pressurized. Partial or total backfilling of the tunnel surrounding the steel conduit with concrete is being considered as an effective defensive measure for this scenario. The tunnel would pressurize in 1 to 10 minutes for flows of 8,000 cfs to 500 cfs, respectively, depending on size of rupture. Once the tunnel has become pressurized, it is unlikely that cavitation damage will continue to occur on a large scale. Some leakage from the pressurized tunnel to the ground surface may occur in the form of springs. However, the potential for seepage from the tunnel to the ground surface through open jointing is remote. It should be easier to repair the RO conduit for the downstream control alternative (once the tunnel can be accessed) than to repair the open channel portion of the conduit for the mid-tunnel or upstream control alternatives. This is because it is simpler to design a transition, for required bends around displacements, for a conduit with pressure flow than it is for a conduit with high velocity, open channel flow. The scenario for the two alternatives, with and without an upstream emergency gate, is discussed below.

(c.1) Without Upstream Emergency Gate. Without the upstream emergency gate the reservoir must be drained before repairs can be made to the conduit. The maximum discharge through the conduit if it separates from the downstream gate structure was estimated to be 12,000 cfs. Effects of discharge on plunge pool and downstream channel were investigated and summarized in a report titled "Channel Stabilization Design and River Sediment Transport Study, Seven Oaks Dam," by Simons, Li and Associates, Incorporated. It was found that the maximum scour depth in the plunge pool would increase from approximately 30 feet for the design discharge of 8,000 cfs to about 50 feet for flows up to 15,000 cfs, and that the downstream channel would armor itself for the larger

discharges. Therefore, the downstream consequence of the maximum uncontrolled discharge of 12,000 cfs is acceptable.

(c.2) <u>With Upstream Emergency Gate</u>. The upstream emergency gate is designed to be either fully open or closed. It is not designed to be a regulating gate and will not be used to maintain positive control of discharge if the conduit ruptures. If the emergency gate is operational, it can be closed and repairs can be made to the conduit without draining the reservoir (assuming there is no major leakage). However, since the possibility of aftershocks is high, it may be several months before the conduit can be safely accessed for repairs. With the upstream emergency gate closed, water would be stored behind the dam. Retaining a high pool is not recommended for the following reasons: if the embankment fails, due to forces from aftershocks and/or previous damage from earthquakes, it would lead to a flood wave causing catastrophic downstream flood damage; if an aftershock occurs with the upstream emergency gate closed, it is possible that this gate will be damaged, making it impossible to drain the reservoir until the upstream emergency gate is repaired; and if the upstream emergency gate jammed while partially open, it would not be possible to quickly drawdown the reservoir, and repairs to the gates or conduit could not be made until the reservoir drained. It is felt that the first priority after a major earthquake should be to drain the reservoir as soon as possible and accept any damage to the outlet works in the process.

(5) <u>Access Failure</u>. This criterion requires that the potential for access failure, as a result of earthquake, be evaluated for each of the alternatives. Access failure occurs when it becomes impossible to reach any critical project feature such as the conduit, gates/control, in order to inspect or repair earthquake damage. In addition, strong aftershocks could cause more conduit damage creating a safety hazard for conduit inspection and repair.

(a) <u>Upstream Control</u>.

(a.1) Gate Access. Access to the gates is by means of a horizontal gallery constructed on top of the outlet conduit. It would be likely that in the event of a 4-foot displacement, the gallery would be partially blocked; reservoir water may enter through a shear at the upstream end, flooding the gallery and blocking access to the upstream gate room. Overbuilt sections could be built at known major shear zones to minimize inflow under some of the displacement scenarios. However, this would not accommodate the displacement occurring at undefined shear planes. Post-earthquake seepage, away from the portals, isn't expected to be of significant concern (see paragraph 5.4.a(11)). Strong aftershocks that could occur for months would also create an unsafe gate access condition.

(a.2) Conduit Access. Access to the conduit is by walking up from the downstream portal. If gates are functioning properly and damage is downstream, then this access is the easiest and best of all the alternatives. However, if, as discussed above, there is some gate failure or significant leakage, then the conduit would be inaccessible. Again, additional displacement and shaking during aftershocks would also cause an access problem.

(a.3) Control Access. The gate controls are located at the downstream end of the outlet conduit. Access to the controls are unlikely to ever be a problem. Because of the 1,700-foot distance between the controls and the gates, however, the control lines for this alternative would be highly vulnerable to displacement damage and perhaps even water flow.

(b) Mid-Tunnel Control.

(b.1) Gate Access. Access to the gates is by means of a vertical shaft and tower with a combined height of nearly 500 feet. Normally, an elevator would convey an inspection party to the gate chamber, and stairs would be available as backup. In the event that the plane of a large (up to 4-foot) displacement intercepts the shaft, the elevator would be left inoperable and the stair damage would have to be

bypassed. If the shearing is in the form of a number of smaller displacements, stair damage should be minimal, allowing access to the gate chamber. If the damage was extensive, access to the gate chamber could be blocked by fallen debris. In addition, should the pressurized portion of the tunnel and/or shaft be ruptured (either tunnel, ground, or surface reservoir water), the gate chamber and the shaft may be flooded and access denied.

(b.2) Conduit Access. The conduit downstream of the gates is accessed by walking up the conduit from the downstream portal. If the gates are undamaged, this remains simple; however, as noted above, the gates of this alternative would be highly vulnerable to damage due to initial and aftershock displacement. Should the gates leak, the conduit would then be inaccessible. There is no provision for conduit access upstream of the gates if the reservoir submerges the intake tower.

(b.3) Control Access. Access to controls in this alternative is likely to be somewhat more difficult than the other alternatives. A tower and bridge arrangement is required as existing ground at the shaft entrance is approximately 50 feet below maximum reservoir pool. The design displacement could seriously damage the tower or bridge. The control lines between the tower and gate chamber will be 500 feet in length and would be highly vulnerable to displacements occurring vertically or horizontally about the access shaft.

(c) Downstream Control.

(c.1) Gate Access. The gate chamber is located at the downstream end of the outlet conduit. Gate access is the easiest with this alternative and is ensured regardless of the design earthquake or any additional tunnel damage due to aftershocks. There could be some difficulty, however, if both the steel pipe and the downstream tunnel plug were to rupture, but with this location access would be best for clearing gate passage for reservoir draining and then implementing repair.

(c.2) Conduit Access. There is no provision for conduit access if the reservoir submerges the intake tower and the upstream

emergency gate is not operational. Once the pool level is below the intake tower maintenance deck, the maintenance bulkhead could be utilized to dewater the steel conduit. Access could then be made through the gate passages from the downstream end.

(c.3) Control Access. The control will be located with the gates at the downstream portal. Access and repair are not foreseen as a problem unless both the tunnel plug and steel conduit rupture.

(6) Downstream Plug Failure. This criterion requires that the potential for the downstream plug to blowout, due to water pressure, and the resulting consequences be evaluated. The downstream control alternative is the only option which has a downstream plug. If the steel conduit ruptures it could flood and pressurize the diversion tunnel and downstream plug. To evaluate a worst case condition we assume that the design earthquake occurs simultaneously with a high pool. Gates partially opened or closed on a rising pool will allow full reservoir pool pressure to develop. Gates opened will significantly reduce potential blowout pressures at downstream end. At full pool (500 feet) the pressure head with the gates opened is approximately 250 feet, which requires 100+ feet of rock cover, while with the gates closed, approximately 200 feet of rock confinement is required. The downstream 150 feet lacks 200 feet of rock cover. Under the future normal debris pool with gates closed, the static head at the downstream portal is 250 feet, for which confinement is adequate. The probability of sustaining damage and potential for blowout is reduced by considering the following defensive measures: the probability of the design earthquake occurring at the same time as a high rising pool is small (see paragraph 5.4.a); post-earthquake seepage is not expected to be significant because of the tightness of the rock mass; the concrete liner can be designed for the expected internal pressures; the downstream 150 feet can be backfilled with concrete to minimize seepage paths; the downstream portal rock can be reinforced through grouting and rock bolting; and internal and external drains may be utilized to provide pressure relief, where external drains might be either horizontal drains at the portal face or parallel tunnel drains used to intercept the seepage. Even if damaged in an earthquake, the residual strength of the

damaged tunnel system (concrete plug, liner, and rock mass) will still be sufficient to prevent a blowout. At worst, leakage will occur, the reservoir would be drained, and repairs made; a catastrophic dam failure scenario is not conceivable for this option.

(7) Tunnel Plug Failure. This criterion requires that the potential for the concrete plug within the tunnel (upstream or downstream of the gate chamber) to rupture, and the resulting consequences be evaluated for each of the alternatives.

(a) Upstream Control. If a significant part of the 4-foot displacement were to occur at the gate plug, flooding of the gate chamber and/or access gallery could occur. As previously discussed, partial or complete loss of positive control could occur.

(b) Mid-Tunnel Control. The mid-tunnel gate plug will be located within the best possible rock mass on the conduit alignment upstream of where it intersects the embankment axis. Selecting the actual location for this site is very restricted by the nearness of the dam axis to the sloping upstream rock face. At best, small displacements (inches) at the plug locations (all alternatives) cannot be guaranteed. Fully confined as it is by the rock mass, such a displacement could heavily damage the plugs at either end of the gate chamber. A rupture at the upstream end would flood the gate chamber and the access shaft.

(c) Downstream Control. The upstream plug for the downstream control is located within the diversion tunnel at the upstream portal. Rupturing of this plug would flood the tunnel access and could precipitate a progressive failure of the steel conduit as discussed above. On the other hand, with a rigid conduit support system, pressurizing the tunnel may not adversely affect the steel conduit.

(8) Intake Tower Failure. This criterion requires that the potential for the intake to be damaged or plugged such that the pool cannot be drained, be evaluated for each alternative. The intake tower for all three alternatives has essentially the same design and geometric

shape. The tower below El. 2,156 is significantly embedded and is considered foundation replacement; the tower above this elevation is circular and treated as a cantilever. As per current Corps structural design criteria (concrete dams and outlet works), the tower is designed using the operating conditions most likely to exist coincident with the selected design earthquake. The tower will perform within the elastic range for the operating basis earthquake (OBE) combined with a normal debris pool. With reduced safety factors, the tower is designed for a maximum credible earthquake (MCE, a=0.7g) in combination with a debris pool or an OBE combined with a 10 year flood. An event with a pool higher than El. 2,350 and an earthquake of acceleration 0.5g or greater, would risk significant damage to the intake tower above El. 2,156 or even tower collapse.

(9) Worst Gate Position. This criterion requires that the worst position for the gates to be stuck at and the consequences of that be evaluated for each of the alternatives. Three gate positions: near full gate opening; mid-range gate opening - 0.75 feet to nearly open; and small gate openings - less than 0.75 feet, are evaluated. Scenarios are similar for each of the three alternatives except that the downstream control alternative may not have an upstream emergency gate. It is assumed that air supply remains functional for each of the evaluations.

(a) Upstream Control.

(a.1) Near Full Gate Opening. If the service gate becomes stuck open at a high pool and the upstream emergency gate can be closed, leakage is not severe, and the gates can still be accessed, then repairs can be made. If there is inflow to reservoir during repairs, flows may pass over the spillway. After repairs to the gate, controlled releases can be made to lower the pool elevation. If the upstream emergency gate cannot be closed, positive control of discharge will be lost. The discharge rate will vary with pool elevation and will be a function of the condition of the RO conduit. The maximum discharge with all gates fully open has been calculated and is not large enough to cause a catastrophic effect downstream. Uncontrolled releases will continue until the pool

drops below the high level intake, El. 2,265. If the wet well sluice gate can be closed or is already closed, and the minimum discharge line is closed, flow, except for leakage, can be stopped until the pool rises above the high level intake. Otherwise, the reservoir will continue to drain until the pool is below the lowest open row of multilevel withdrawal ports. If the regulating conduit has been damaged, the uncontrolled discharge may cause erosion and/or damage to the conduit.

(a.2) Mid-Range Gate Opening. The scenario is the same as above except that uncontrolled discharges may be less so it could take longer for the reservoir to drain to a pool level at which repairs could be made. If larger discharges are required, it may be possible to open the gates for the low flow bypass and minimum discharge line.

(a.3) Small Gate Opening. The scenario is the same as for the other two cases except that it may take much longer for the reservoir to drain and significant cavitation damage may occur to the gate if it is stuck at a small gate opening when the pool elevation is high. This is the worst case.

(b) Mid-Tunnel Control. The scenario if the gates are stuck is the same as described for upstream control.

(c) Downstream Control. The scenario if the gates are stuck is the same as described for upstream control except for the redundancy provided by the upstream emergency gate (see paragraph 5.5.1.c) and access to make repairs to the gates is better.

(10) Air Supply Failure. This criterion requires that the likelihood and consequences of a blockage in, or failure of, the air supply vents be evaluated for each alternative.

(a) Upstream Control. The air passageway located in the horizontal tunnel has an intake at the downstream end of the tunnel and supplies air to the vents above the aeration offsets near the gates. The passageway will probably be damaged during an earthquake. If the air

supply system does not function as designed, there will be cavitation damage at the offsets and there will likely be cavitation damage to the gates and conduit downstream of the gates, which could lead to loss of positive control and expensive repairs. At large gate openings, slug flow may also occur which could create damaging wave action in the downstream channel.

(b) Mid-Tunnel Control. The scenario is the same as described above except that the air supply is more likely to fail than for the upstream control alternative since the vertical shaft is unreinforced and all loose material will funnel downward, blocking the air passage.

(c) Downstream Control. The air supply system for the downstream control alternative has an excellent chance of surviving an earthquake since the air vents are located directly above the offsets and open to the atmosphere. Even in the unlikely event that the air vents are plugged, air drawn from open areas downstream will most likely provide enough air to prevent severe cavitation until the vents can be cleared. If cavitation damage does occur it is not likely to progress very far downstream, it will not cause a catastrophic failure, and will probably not lead to loss of positive control of discharge.

(11) Rock Mass Seepage. This criterion requires that the likelihood and consequences of seepage of water along a shear plane induced by new or reactivated faulting be evaluated for each alternative. Water pressure testing in core holes indicates a generally low permeability for the rock mass, indicating that seepage should not be a significant problem. Because the rock mass is believed to be in slight compression, it is likely that fault rupturing would not create a shear plane capable of transmitting a significant seepage volume. Also, since the bedrock has been sheared numerous times and has remained tight, it is expected that further rupture from the postulated design earthquake will not create seepage between the pool and the tunnel along the rupture zone. Field investigations on this and other projects in similar geologic environment indicate shear zones are generally tight and actually become effective barriers to seepage. The consequences of seepage from the

reservoir to the tunnel would probably be similar for all three alternatives. Because of the additional potential for seepage to enter the tunnel presented by the vertical shafts, however, the mid-tunnel control alternative would be somewhat more vulnerable. With the downstream control alternative, the tunnel would be pressurized and water could be forced into the rock mass at the rupture point, especially if it occurred downstream of the grout curtain where seepage pressure would not be counteracted by hydrostatic pressure in the rock. Due to limited rock permeability, this leakage would not be a significant problem. Tunnel location relative to embankment is shown in figure 5-1.

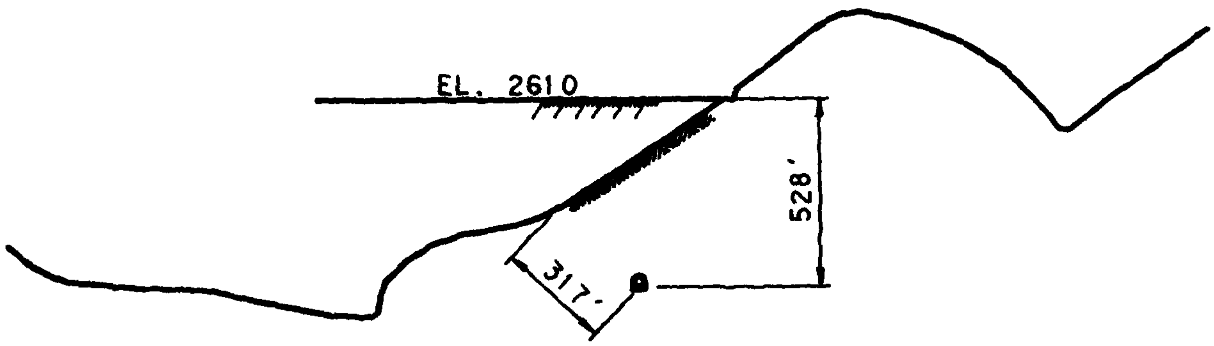
b. Construction Cost Estimate Comparison.

(1) Construction Cost Summary. A summary of key cost items for the three outlet alternatives is given in table 5-2.

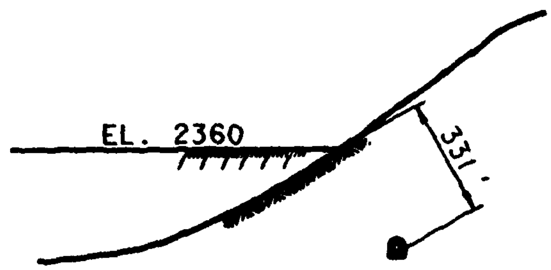
Table 5-2. Alternative Cost Estimates Summary (in millions $)

	Alternatives		
Item	Upstream	Mid-tunnel	Downstream
Project roads	1.8	1.9	1.8
Intake excavation	2.5	2.5	2.5
Tunnel/shaft excavation	7.4	8.4	4.3
Outlet channel excavation	1.9	2.0	2.1
Concrete	10.4	8.8	7.4
Metals and structural steel	0.3	0.9	6.2
Mechanical and Electrical	2.3	2.6	2.2
Miscellaneous plus 15 percent contingency	6.7	6.9	6.8
SUB-TOTAL	33.3	34.0	33.5
Additional features:			
Upstream emergency gate	---	3.6	3.6
Seismic measures	+3%	+3%	+3%
TOTAL CONSTRUCTION COSTS	34.3	38.7	38.2

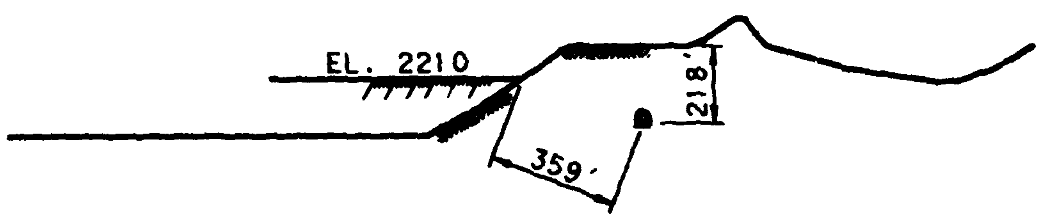
RELATIVE TUNNEL/EMBANKMENT LOCATION



TUNNEL STA. 18+60
SCALE: 1" = 500'



TUNNEL STA. 23+00
SCALE: 1" = 500'

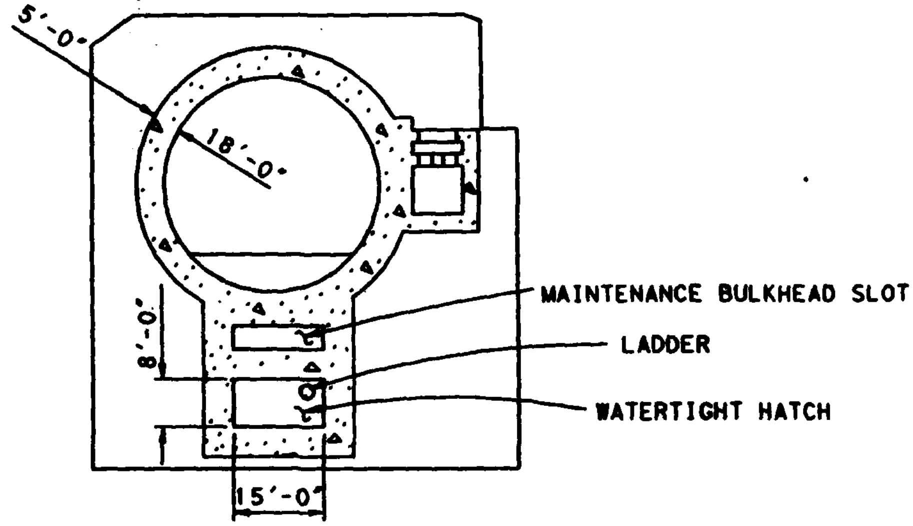


TUNNEL STA. 26+00
SCALE: 1" = 500'

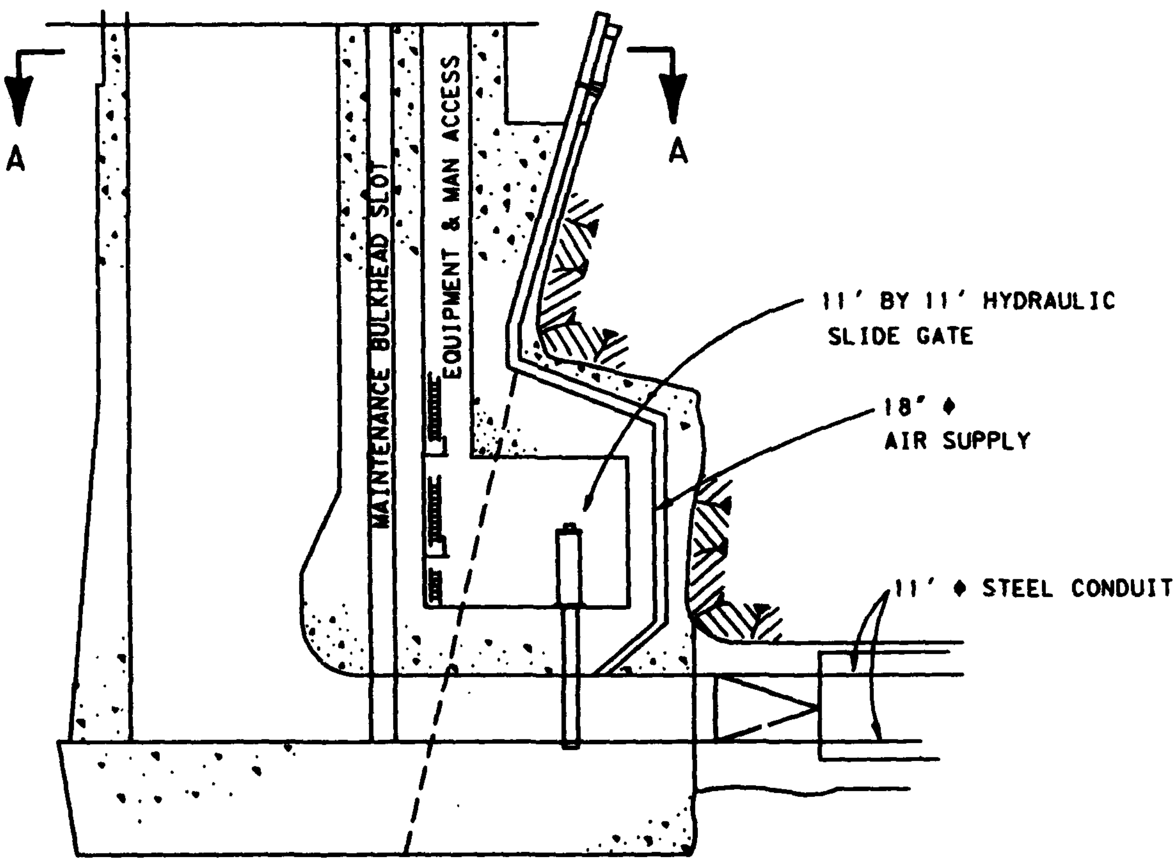
Figure 5-1

Detailed construction cost estimates for the outlet works alternatives are shown in tables 5-6, 5-7, and 5-8, pages 5-35 through 5-44. All costs are based on March 1988 price levels. Unit costs are derived from recent bid costs on similar work and historical cost data. Prior to adding costs for additional "dam safety" measures, the three alternatives are essentially the same at $34 million each. The mid-tunnel and downstream control alternatives become the most expensive at $38 million when the cost of emergency upstream gating is included (see figure 5-2). At the January TRC, OCE representatives directed that an upstream emergency gate would be required for any downstream alternatives. This direction came about from a concern that a high pool would exist coincidental with the design earthquake (4-foot displacement). Under this scenario, uncontrolled releases occur which would prevent inspection of the tunnel and outlet features immediately following the seismic event. The same scenario exists for the mid-tunnel option, and as such the extra upstream gate cost is shown for this measure. No consideration was given to the probability of these independent events occurring at the same time or the associated consequences of the uncontrolled releases. For further discussion of uncontrolled releases (within tunnel and downstream channel) see paragraph 5.4.a(4). For estimating purposes, an upstream gate chamber with an 11-foot square hydraulic slide gate was assumed. Equipment and personnel access would be from the tower maintenance deck at El. 2,270. Seismic design measures (costs) were approximated at 3 percent for all options. The 3 percent accounts for general measures not yet quantified which may be required to satisfy structural or geotechnical design needs (tower embedment, shear zone treatments, displacement scenarios, etc.).

(2) Potential for Cost Growth. This criterion evaluates which alternative has the most uncertainty in the cost estimate and the most complexities for construction. From the geotechnical standpoint, the alternative with the most potential for cost growth is mid-tunnel control. Not only are there significant uncertainties in the cost estimate for constructing and supporting the enlarged tunnel section for the control structures, there are added uncertainties in the cost because this alternative also requires construction of a vertical shaft or shafts. If unforeseen problems were to develop in excavation or support



SECTION A-A



DOWNSTREAM CONTROL W/ U/S EMERGENCY GATE

FIGURE 5-2

of these features, costs could increase significantly. Because tunnel construction is least complex for downstream control, this alternative has the least potential for cost growth from the geotechnical standpoint.

5.5 Evaluation Matrix. At the January 1988 TRC, the idea of using an evaluation matrix was presented. A matrix is shown in table 5-3. This matrix summarizes the evaluation of the three alternatives based on what is considered to be the most significant criteria and factors as described in paragraph 5.3. The comparisons are considered generally relative to each other. Detailed discussion is provided in paragraph 5.4, "Evaluation of Principal Criteria."

Table 5-3. Alternative Evaluation Matrix Reduced to Key Evaluation Criteria Only

([Key: + (best) o (mid) - (worst)])

Factor/Criteria	Upstream Control	Mid-Tunnel Control	Downstream Control
a. EQ Survivability (dam safety)			
(1) Control/gate chamber	o	-	+
(2) Gates (shaking)	o	o	o
(3) Gates (displacement)	-	-	+
(4) Conduit/tunnel	o	o	o
(5) Access			
gate	-	-	+
conduit/tunnel	o	-	-
control	o	-	+
(6) Downstream plug	o	o	o
(7) Tunnel plug	-	-	-
(8) Intake tower	o	o	o
(9) Worst gate position	o	o	+
(10) Air supply failure	-	-	+
(11) Rock seepage	+	-	-
b. Construction cost	+	o	o
c. O&M	o	o	o

5.6 Evaluation of Secondary Criteria. Criterion which are considered to be of a secondary nature, relative to the selection of the outlet works alternative, are evaluated for the three alternatives. These criteria include secondary cost items, constructibility, and general project operability (O&M).

a. Secondary Cost Item Evaluation.

(1) Operation Cost. This criterion evaluates which alternative is the least costly to operate. There are no significant cost differences in operation of the three alternatives. The downstream control is probably the easiest to operate due to the siting of controls and gates in one location and the ease of access. Basically, all three alternatives have essentially the same system gating and operating criteria. See section 11 for summary of typical operating costs.

(2) Maintenance Costs. This criterion evaluates which alternative is least costly to maintain. Each of the alternatives have maintenance differences specific to their features. The actual cost differences are not significant relative to the final selection. Some of the maintenance differences are as follows:

(a) Upstream Control. One thousand six hundred and fifty feet of air conduit and mechanical and electrical duct work within the access adit. All outlet flows use an 18-foot-wide concrete channel, increasing concrete wear and maintenance potential. A downstream gate control and access structure with air intake grating will need to be maintained.

(b) Mid-Tunnel Control. The additional bridge, tower/control structure, air shaft, access shaft, and road will require maintenance. Access shaft has elevator and steel stairs requiring maintenance and painting cycles. Low flow bypass will require a within tunnel trashrack requiring tunnel dewatering for debris removal, replacement, and/or painting. Tunnel channel passes all outlet flows, potential concrete lining maintenance. Shaft electrical and mechanical duct work will require maintenance in a potentially moist environment.

(c) Downstream Control. The exterior of the steel conduit will require a maintenance painting cycle. Potential for greater tunnel adit maintenance, drain, and gutter cleanout.

From a maintenance standpoint, it appears that upstream and downstream control would have similar maintenance costs while mid-tunnel control would have the greatest.

b. Construction Evaluation.

(1) Diversion Plan. This criterion evaluates which alternative has the simplest, most reliable, and highest level of protection in its diversion plan. All three alternatives require complex sequencing of construction through the gate chamber and other appurtenant structures. Because of its location within the heart of the rock mass, the mid-tunnel alternative's gate chamber will be very difficult to construct while simultaneously handling diversion flows. The downstream control has diversion bypass built into the design, but construction sequencing through the gate chamber will still be similar to the other alternatives; with anchoring blockouts and second stage concrete placements are typical for all alternatives. In the final evaluation the upstream control alternative has a slight edge over the others due to its potential for extra capacity, if the access gallery construction is delayed. Otherwise the three alternative are considered essentially equal with regards to diversion advantages and disadvantages.

(2) Tunnel/Shaft Excavation. This criterion evaluates the relative uncertainties in the geotechnical feasibilities of performing the tunnel, shaft, and portal excavations.

(a) Upstream Control. The most significant geotechnical uncertainty with this alternative is for excavating and supporting an enlarged section of tunnel at the upstream portal. Because of the size and shape of the opening, maintaining stability of the upstream portal during construction would be more difficult than for the other control alternatives. This alternative also requires the largest cross section for the main tunnel, making it less desirable from that standpoint.

(b) Mid-Tunnel Control. The principal uncertainty for this alternative is constructibility of the enlarged tunnel cross section or "chamber," where the intersection with the vertical shaft(s) occurs. Underground openings of this size in questionable quality rock should be avoided, if possible. Chamber excavation and support would require closely controlled methods and sequencing. The additional required construction of one or two vertical shafts adds somewhat to the geotechnical concerns for this alternative; however, it is believed that shafts up to about 15 feet in diameter can be satisfactorily constructed using the raise bore method.

(c) Downstream Control. No significant geotechnical uncertainties regarding constructibility exist with the downstream control alternative.

c. Operability (Operations and Maintenance) Evaluation. This criterion compares the alternatives with respect to their differences in operability, reliability, and maintenance characteristics.

(1) Reliability of Slide Gates Under High Heads. Regulating outlets (RO) slide gates on several dams built by the Corps of Engineers, the Bureau of Reclamation, and British Columbia Hydro and Power Authority (BC Hydro) have operated successfully under heads between 360 feet and 500 feet. The RO works at Pine Flat Dam (Corps of Engineers) have RO slide gates that have operated frequently at heads up to 380 feet. No problems with gate operation were encountered. At Glen Canyon Dam (Bureau of Reclamation), slide gates were used as an interim control in a partially plugged diversion tunnel while the dam was under completion. These slide gates operated successfully for approximately 2 years under heads up to 360 feet. At Mica Dam (BC Hydro), slide gates were used on the low level outlets for a period of 3 years to regulate flow while the reservoir was being filled. The gates performed very well at heads up to 500 feet. Slide gates at the upstream and downstream ends of the low level outlet tunnel regulated the flow. Even though the flow was under pressure between the upstream and downstream gates, a low pressure zone occurred immediately downstream of the upstream gate, and personnel at BC Hydro

stated that the differential head on the upstream gate was quite close to the full energy upstream of the gate. An article titled "High-Pressure Outlets, Gates, and Valves," by W. Kohler and J. Ball in the book titled "Handbook of Applied Hydraulics," by Davis and Sorenson, 3rd Ed., states "there appears to be no definite size or head limitation for correctly designed slide gates. The successful use of such gates with only minor cavitation damage at heads of nearly 350 feet at Glen Canyon indicates that 500-foot heads are not unreasonable and that possibly considerably higher heads can be used." The author states that to better resist cavitation damage, the fluidway surfaces, bottom seating, and sloping surfaces of the gate leaf should preferably be stainless steel. With proper design and construction, slide gates should prove safe and reliable for application in the upstream, downstream, and mid-tunnel control alternatives. Table 5-4 lists seven dams in which slide gates have been operated with static heads over 300 feet.

(a) Upstream Control. With this alternative the selected gate design and slide gates will experience a maximum static head of 504 feet with a pool elevation of 2,604 (PMF event). See table 5-5 for head and duration data.

(b) Mid-Tunnel Control. With this alternative the selected gate design and slide gates will experience a maximum static head of 513 feet at the PMF event. See table 5-5 for head and duration data.

(c) Downstream Control. With this alternative the selected gate design and slide gates will experience a maximum static head of 545 feet at the PMF event. See table 5-5 for head and duration data.

(2) Gate Maintenance. The three alternatives all have a similar gating system; four 5-foot by 9-foot hydraulic slide gates, two 2-foot by 3.5-foot low flow slide gates, minimum discharge gating, and a wet well sluice gate. The only real difference in gate maintenance is in access and ease for making repairs. The downstream control alternative offers the best location with respect to gate maintenance.

Table 5-4. Summary of High Head Dams with Slide Gates.

Project	Head On Gate (feet)	Comments
Detroit	*305	Gates have not operated since 1956. Gates were used before turbines went on line. CORPS
Pine Flat	**381	Gates have operated frequently under this head. No problems. CORPS
Carters	*349 **400	Gates are for emergency operation and have not operated since construction. Head @ 349 feet for short time. CORPS
Mica Dam	*450-500 **570	Gates were operated for 3 years with no problems. Gates are no longer used. BC HYDRO
Glen Canyon	*360	Gates were operated frequently at high heads for 2 years. No operational problems. USBR
Morrow Point	**400	Gates operated with no problems, gates are not operated as frequently as those at Glen Canyon. USBR
Palisades Dam	*346	No problems at the gates. USBR

* Maximum head gate operated under
**Design head

Table 5-5. Future Conditions - Static Head on Regulating Outlet Slide Gates

Pool El. (Ft)	Cumulative Duration (Days)**	*STATIC HEAD (FT) U/S Control	D/S Control	M-T Control
2,325	408	225	265	235
2,350	231	250	290	260
2,400	89	300	340	310
2,450	47	350	390	360
2,500	15	400	440	410
2,550	2	450	490	460

* Static Head Equals Pool El. Minus Invert at Gate
**Cumulative days at or above pool elevation over 100 years, after 100-year deposition (165 feet) worst case

(3) Conduit/Tunnel Maintenance.

(a) Upstream Control. This alternative has a lengthy adit requiring maintenance of relief drains and mechanical and electrical equipment. There is also potential for tunnel channel erosion, abrasion, or cavitation damage depending on outlet releases and air demand.

(b) Mid-Tunnel Control. Tunnel channel wear and tear will be similar to upstream control. This alternative has a low flow trashrack located in the pressurized portion of the tunnel immediately upstream of the gate chamber. This trashrack will require tunnel dewatering to clean, paint, or replace trashrack components. Access will be through the 5-foot by 9-foot slide gates.

(c) Downstream Control. For downstream control the concrete channel is replaced with a steel conduit. As a pressure conduit the conduit isn't expected to be subjected to cavitation potential. The steel thickness will be sized for loading stresses and abrasion protection. The tunnel is actually expected to remain relatively dry most of the time. This is due to the tightness of the rock and the operating plan (flood control) for the reservoir ("dry reservoir"). The 11-foot-diameter conduit itself will have minimal use through its lifetime (except during flood events), as the low flow and minimum discharge pipes will carry most of the normal operating discharges. A normal painting cycle of 10 years is predicted for the exterior of the steel conduit. In the accessible portion of the tunnel, gutters and drains will require periodic maintenance. The differing maintenance activities of the three alternatives are not considered significant with respect to the alternative selection.

(4) Tower Mainenance Activities. A similar tower arrangement has been designed for all three of the alternatives. Trash and debris removal, trashrack maintenance, stoplogs, metalwork, and bulkhead maintenance will be similar for all three alternatives. The downstream alternative will have a low flow entrance, trashrack, and provisions for

maintenance bulkheading located at the bottom of the large wet well. This would be the only difference between alternatives relative to tower maintenance activities. This difference is not considered significant.

5.7 Construction Schedule. Construction of the outlet works will be completed in two construction phases; a tunnel contract and the embankment dam contract. The first phase provides diversion for the embankment dam construction. To accommodate future outlet works construction, some of the outlet works structures are partially completed during the first phase. The first phase is completed in approximately 16 months. The second phase follows immediately, and will continue for approximately 5 years. Once the embankment has reached an SPF level of protection, the intake tower and other remaining outlet structures will be completed. This schedule is essentially the same for all three alternatives, with some sequencing differences as noted in the following schedule summaries (see figures 5-3 through 5-5).

a. Upstream Control. Outlet/diversion tunnel construction is planned to start with the downstream portal excavation followed by drill and blast tunnel excavation from the downstream heading. Due to the 35-foot height of this tunnel, two bench construction will be required. Tunnel support (rock anchors, ribsets, and shotcrete) will follow directly behind the excavation. The concrete liner will be formed with 40-foot sections of steel form. Contact grouting and drains will complete the tunnel work. If increased diversion capacity is needed, it is feasible to delay and install the tunnel adit floor during the embankment dam contract. The upstream portal excavation will provide the foundation for the intake tower and gate chamber, and will be constructed concurrently with the downstream portal excavation. A portion of the tower base and gate chamber will be constructed with appropriate blockouts, falsework, and separator piers to pass expected diversion flows. During embankment construction river flow will be diverted through the 18-foot-wide tunnel. Completion of the tower gate chamber, downstream structures, gate installation, and other outlet works features will be performed by the embankment contractor. Once the embankment reaches SPF level of protection, the tower and related structures will be completed during the

dry season and full project benefits will be realized. Diversion through the control section will be accomplished by blocking out the gate areas and then sequencing gate installation and concrete placement during the summer low flow period.

b. Mid-Tunnel Control. To maintain the same schedule for all alternatives, two full headings are required for the mid-tunnel alternative. This will allow for an earlier start at the raised bore mining of the access shaft, and will prevent the mucking operations from interfering with each other. All alternatives will require some minimal flood protection for the upstream portal construction. With the downstream heading, exposure and duration are increased, thus, additional protection may be required. Tunnel and shaft excavation will be complete prior to the excavation of the expanded mid-tunnel gate chamber. Diversion flows will be passed through the partially completed gate chamber during the embankment contract similar to the upstream control alternative. Completion of the shaft, tower(s) and other outlet features will be done by the embankment contractor, primarily around the time the embankment reaches the SPF level of protection.

c. Downstream Control. Tunnel excavation will start from the downstream portal on an upstream heading. Upstream and downstream structures will be partially completed similar to the alternatives described above. For downstream control the diversion will be through a partially completed intake, the 18-foot-wide tunnel, and a blocked out passage in the downstream gate structure. When the embankment reaches the SPF level the second phase features will be installed. Dry season diversion flows will be diverted through small pipes located in the floor of the tunnel. The steel conduit and supports, tower, and other remaining outlet features will be completed during this summer low flow period. While the small pipe gate system is being completed, water will be diverted through the RO conduit.

5.8 Outlet Works Recommendation. The downstream control alternative is the recommended outlet works system for The Seven Oaks Dam project. This alternative offers the best system for earthquake survivability. It also

assumes a better assurance that there will be no tunnel blockage, and the gates and controls are readily accessible for positive drawdown of the reservoir. General operability is the best, and costs are essentially the same as the mid-tunnel control and about 10 percent greater than upstream control. All the alternatives were considered feasible and could be acceptable within Corps standards. With consideration of the final evaluation of the principal criteria, the alternatives were ranked as follows: downstream control; upstream control; and mid-tunnel control.

TABLE 5-6

Santa Ana Project - Seven Oaks Outlet Works - Cost Estimate

Upstream Control - Downstream Access - High Level Intake Tower

29-Jul-88

Item	Unit	Quantity	Unit Cost	Item Cost	Subtotal
1. Mobilization	LS	1	$2,000,000.00	$2,000,000	
					$2,000,000
2. Clearing and Grubbing	AC	10	$800.00	$8,000	
					$8,000
3. Diversion and Cofferdams	LS	1	$200,000.00	$200,000	
					$200,000
4. Project Roads					
a. Intake access road	LF	4,500	$80.00	$360,000	
(1) rockbolts	LF	7,200	$15.00	$108,000	
(2) excavation	CY	43,000	$15.00	$645,000	
(3) mesh	SF	36,000	$3.25	$117,000	
(4) backfill	CY	15,000	$5.00	$75,000	
b. D/S access road	LF	1,500	$80.00	$120,000	
(1) rockbolts	LF	1,100	$15.00	$16,500	
(2) excavation	CY	47,000	$5.00	$235,000	
(3) rock excavation	CY	9,000	$15.00	$135,000	
(4) mesh	SF	3,500	$3.25	$11,375	
(5) safety fence	LF	450	$25.00	$11,250	
					$1,834,125
5. Excavation (Intake)					
a. Overburden	CY	80,000	$5.00	$400,000	
b. Rock	CY	70,000	$15.00	$1,050,000	
c. Foundation Prep.	SY	9,700	$40.00	$388,000	
d. Slope Treatment					
(1) rockbolts	LF	21,000	$15.00	$315,000	
(2) shotcrete	CY	820	$300.00	$246,000	
(3) fencing	LF	900	$25.00	$22,500	
(4) consolidation grout	LS	1	$100,000.00	$100,000	
					$2,521,500
6. Excavation (Preformed Plunge Pool)					
a. Overburden	CY	50,000	$5.00	$250,000	
b. Foundation Backfill	CY	3,000	$5.00	$15,000	
c. Riprap	CY	2,000	$55.00	$110,000	
d. Slope Treatment					
(1) tiebacks	LF	7,000	$15.00	$105,000	
					$480,000
7. Excavation (Outlet Portal)					
a. Overburden	CY	105,000	$5.00	$525,000	
b. Rock	CY	37,000	$15.00	$555,000	
c. Foundation Prep.	SY	3,000	$40.00	$120,000	
d. Slope Treatment					
(1) rockbolts	LF	8,000	$15.00	$120,000	
(2) shotcrete	CY	280	$300.00	$84,000	
(3) mesh	SF	2,600	$3.25	$8,450	
(4) fencing	LF	800	$25.00	$20,000	
					$1,432,450

Santa Ana Project - Seven Oaks Outlet Works - Cost Estimate
Upstream Control - Downstream Access - High Level Intake Tower

Item	Unit	Quantity	Unit Cost	Item Cost	Subtotal
8. Tunnel Excavation (Oval W/ Drains)					
a. Excavation	CY	51,000	$100.00	$5,100,000	
b. Support					
(1) shotcrete	CY	2,550	$300.00	$765,000	
(2) rockbolts	LF(Tunnel)	1,900	$30.00	$57,000	
(3) drains	LF	600	$50.00	$30,000	
(4) ribs	LF(Tunnel)	1,900	$450.00	$855,000	
c. Contact Grouting	LS	1	$500,000.00	$500,000	
d. Grout ring	LS	1	$100,000.00	$100,000	
					$7,407,000
9. Instrumentation					
a. Geotechnical	LS	1	$100,000.00	$100,000	
b. Hydraulic	LS	1	$25,000.00	$25,000	
					$125,000
10. Concrete					
a. Intake Structure					
(1) below elev. 2156.0	CY	9,000	$225.00	$2,025,000	
(2) above elev. 2156.0	CY	4,600	$225.00	$1,035,000	
b. U/S Access bridge	LS	1	$60,000.00	$60,000	
c. Oval Tunnel lining	CY	18,860	$350.00	$6,601,000	
d. D/S Access Structure	CY	800	$225.00	$180,000	
e. Exit Channel Wall	CY	1,400	$225.00	$315,000	
f. Cutoff wall	CY	600	$225.00	$135,000	
					$10,351,000
11. Miscellaneous metals					
a. Handrails and ladders	LS	1	$40,000.00	$40,000	
b. Low flow piping	LB	12,000	$0.50	$6,000	
c. Grating and hatches	LS	1	$50,000.00	$50,000	
					$96,000
12. Structural steel					
a. RO bulkhead & guides	LB	35,000	$3.30	$115,500	
b. Trashracks	LS	1	$10,000.00	$10,000	
c. Air duct for gates	LS	1	$25,000.00	$25,000	
					$150,500
13. Mechanical Equipment					
a. Floatwell mechanisms	LS	1	$30,000.00	$30,000	
b. Water supply	LS	1	$30,000.00	$30,000	
c. Drains	LS	1	$15,000.00	$15,000	
d. RO gates, frames, cylinders & operators	LS	1	$1,500,000.00	$1,500,000	
e. Fuel tank & generator	LS	1	$70,000.00	$70,000	
f. Sanitary Facilities	LS	1	$10,000.00	$10,000	
g. Heating & ventilating	LS	1	$85,000.00	$85,000	
h. Access vehicle	LS	1	$10,000.00	$10,000	
i. 2' x 3.5' gating	LS	1	$60,000.00	$60,000	
j. Hoists	LS	1	$70,000.00	$70,000	
k. Sluice gate (manual)	LS	1	$40,000.00	$40,000	
l. low flow gating/piping	LS	1	$40,000.00	$40,000	
					$1,960,000

Santa Ana Project - Seven Oaks Outlet Works - Cost Estimate
Upstream Control - Downstream Access - High Level Intake Tower

Item	Unit	Quantity	Unit Cost	Item Cost	Subtotal
14. Electrical Equipment	LS	1	$300,000.00	$300,000	
					$300,000
15. Architectural Features	LS	1	$100,000.00	$100,000	
					$100,000
Sub-Total Const. costs		$28,965,575			$28,965,575
Contingency	15.0%	$4,344,836			
Total Construction Costs Upstream Control		$33,310,411			

TABLE 5-7

Santa Ana Project - Seven Oaks Outlet Works - Cost Estimate

Mid-tunnel Control - Shaft Access - High Level Intake Tower 29-Jul-88

Item	Unit	Quantity	Unit Cost	Item Cost	Subtotal
1. Mobilization	LS	1	$2,100,000	$2,100,000	
					$2,100,000
2. Clearing and Grubbing	AC	10	$800	$8,000	
					$8,000
3. Diversion and Cofferdams	LS	1	$300,000	$300,000	
					$300,000
4. Project Roads					
a. Intake access road	LF	4,500	$80	$360,000	
(1) rockbolts	LF	7,200	$15	$108,000	
(2) excavation	CY	43,000	$15	$645,000	
(3) mesh	SF	36,000	$3.25	$117,000	
(4) backfill	CY	15,000	$5	$75,000	
b. D/S access road	LF	1,500	$80	$120,000	
(1) rockbolts	LF	1,100	$15	$16,500	
(2) excavation	CY	47,000	$5	$235,000	
(3) rock excavation	CY	9,000	$15	$135,000	
(4) mesh	SF	3,500	$3.25	$11,375	
(5) safety fence	LF	450	$25	$11,250	
c. Shaft access road (spillway access road)					
(1) bridge abutment & landing	LS	1	$25,000	$25,000	$1,859,125
5. Excavation (Intake)					
a. Overburden	CY	80,000	$5	$400,000	
b. Rock	CY	70,000	$15	$1,050,000	
c. Foundation Prep.	SY	9,700	$40	$388,000	
d. Slope Treatment					
(1) rockbolts	LF	21,000	$15	$315,000	
(2) shotcrete	CY	820	$300	$246,000	
(3) fencing	LF	900	$25	$22,500	
(4) consolidation grout	LS	1	$100,000	$100,000	
					$2,521,500
6. Excavation (Preformed Plunge Pool)					
a. Overburden	CY	59,200	$5	$296,000	
b. Foundation Backfill	CY	3,000	$5	$15,000	
b. Riprap	CY	2,000	$55	$110,000	
c. Slope Treatment					
(1) tiebacks	LF	7,000	$15	$105,000	
					$526,000

Santa Ana Project - Seven Oaks Outlet Works - Cost Estimate
Mid-tunnel Control - Shaft Access - High Level Intake Tower

Item	Unit	Quantity	Unit Cost	Item Cost	Subtotal
7. Excavation (D/S exit channel)					
a. Overburden	CY	105,000	$5	$525,000	
b. Rock	CY	37,000	$15	$555,000	
c. Foundation Prep.	SY	3,000	$40	$120,000	
d. Slope Treatment					
(1) rockbolts	LF	8,000	$15	$120,000	
(2) shotcrete	CY	280	$300	$84,000	
(3) mesh	SF	2,600	$3.25	$8,450	
(4) fencing	LF	800	$25	$20,000	
					$1,432,450
8. Tunnel Excavation (Horseshoe 11+30 to 14+83)					
a. Excavation	CY	8,350	$100	$835,000	
b. Support					
(1) shotcrete	CY	450	$300	$135,000	
(2) rockbolts	LF(Tunnel)	353	$30	$10,590	
(3) drains	LF	2,000	$50	$100,000	
(4) ribs	LF(Tunnel)	353	$350	$123,550	
c. Contact Grouting	LS	1	$500,000	$500,000	
					1,704,140.00
9. Tunnel Excavation (control section 14+83 to 16+90)					
a. Excavation	CY	9,025	$150	$1,353,750	
b. Support					
(1) shotcrete	CY	360	$300	$108,000	
(2) rockbolts	LF	207	$450	$93,150	
(3) drains	LS	1	$75,000	$75,000	
(4) ribs	LF	207	$5,000	$1,035,000	
c. Contact Grouting	LF	207	$700	$144,900	
					2,809,800.00
10. Tunnel Excavation (Horseshoe 16+90 to 27+57)					
a. Excavation	CY	17,685	$100	$1,768,500	
b. Support					
(1) shotcrete	CY	1,070	$300	$321,000	
(2) rockbolts	LF	1,067	$30	$32,010	
(3) ribs	LF	1,067	$325	$346,775	
(4) grout ring	LS	1	$20,000	$20,000	
c. Contact Grouting	LF	1,067	$350	$373,450	
					2,861,735.00
11. Shaft Excavation (15 ft ID)					
a. Excavation	CY	3,850	$115	$442,750	
b. Support					
(1) shotcrete	CY	440	$300	$132,000	
(2) rockbolts	LF(shaft)	406	$30	$12,180	
(3) drains	LF(shaft)	406	$200	$81,200	
(4) contact grout	LS			0.00	
					668,130.00

Santa Ana Project - Seven Oaks Outlet Works - Cost Estimate
Mid-tunnel Control - Shaft Access - High Level Intake Tower

Item	Unit	Quantity	Unit Cost	Item Cost	Subtotal
12. Shaft Excavation (10 ft ID)					
a. Excavation	CY	1,450	$100	$145,000	
b. Support					
(1) shotcrete	CY	275	$300	$82,500	
(2) rockbolts	LF	406	$30	$12,180	
(3) drains	LF	406	$200	$81,200	
					320,880.00
13. Instrumentation					
a. Geotechnical	LS	1	$100,000	$100,000	
b. Hydraulic	LS	1	$25,000	$25,000	
					$125,000
14. Concrete					
a. Intake Structure					
(1) Below el. 2156.0	CY	8,700	$200	$1,740,000	
(2) Above el. 2156.0	CY	4,600	$400	$1,840,000	
b. U/S Access bridge	LS	1	$60,000	$60,000	
c. Shaft access bridge	LS	1	$60,000	$60,000	
d. Exit Channel & Bucket	CY	1,400	$225	$315,000	
e. Tunnel u/s	CY	4,070	$250	$1,017,500 STA 11+30 TO 14+83	
f. Tunnel d/s	CY	5,345	$250	$1,336,250 STA 16+90 TO 27+57	
g. Shafts and towers	CY	3,875	$250	$968,750	
h. Mid-tunnel control	CY	5,925	$250	$1,481,250 STA 14+83 TO 16+90	
i. Cutoff wall	CY	500	$225	$112,500	
					$8,818,750
15. Miscellaneous metals					
a. Handrails and ladders	LS	1	$60,000	$60,000	
b. Grating and hatches	LS	1	$50,000	$50,000	
c. Shaft stairs	LS	1	$150,000	$150,000	
					$260,000
16. Structural steel					
a. RO bulkhead & guides	LB	120,000	$3.30	$396,000	
b. Tower trashracks	LS	1	$20,000	$20,000	
c. Min. disch. conduit	LS	1	$20,000	$20,000	
d. Low flow trashrack	LS	1	$20,000	$20,000	
e. RO lining	LB	65,000	$1.90	$123,500	
f. RO gating air ductwork	LS	1	$10,000	$10,000	
					$589,500

Santa Ana Project - Seven Oaks Outlet Works - Cost Estimate
Mid-tunnel Control - Shaft Access - High Level Intake Tower

Item	Unit	Quantity	Unit Cost	Item Cost	Subtotal
17. Mechanical Equipment					
a. Floatwell mechanisms	LS	1	$30,000	$30,000	
b. Water supply	LS	1	$30,000	$30,000	
c. Structure Drains	LS	1	$15,000	$15,000	
d. RO gates,frames, cylinders & operators	LS	1	$1,500,000	$1,500,000	
e. Fuel tank & generator	LS	1	$70,000	$70,000	
f. Sanitary Facilities	LS	1	$10,000	$10,000	
g. Heating & ventilating	LS	1	$20,000	$20,000	
h. 2'x 3.5' gating	LS	1	$120,000	$120,000	
i. Hoists	LS	1	$70,000	$70,000	
j. U/S Min flow bkhd gate	LS	1	$125,000	$125,000	
k. Elevator	LS	1	$250,000	$250,000	
l. Min. discharge valves	LS	1	$50,000	$50,000	
k. Min. disch. piping	LF	450	$50	$22,500	
					$2,312,500
18. Electrical Equipment	LS	1	$275,000	$275,000	
					$275,000
19. Architectural Features	LS	1	$100,000	$100,000	
					$100,000
Sub-Total Const. costs		$29,592,510			$29,592,510
Contingency	15.0%	$4,438,877			
Total Construction Costs Mid-Tunnel Control		$34,031,387			

TABLE 5-8

Santa Ana Project - Seven Oaks Outlet Works - Cost Estimate

Downstream Control - Steel RO Conduit - High Level Intake Tower 29-Jul-88

Item	Unit	Quantity	Unit Cost	Item Cost	Subtotal
1. Mobilization	LS	1.00	$2,000,000	$2,000,000	
					$2,000,000
2. Clearing and Grubbing	AC	10.00	$800	$8,000	
					$8,000
3. Diversion and Cofferdams	LS	1.00	$300,000	$300,000	
					$300,000
4. Project Roads					
a. Intake access road	LF	4,500.00	$80	$360,000	
(1) rockbolts	LF	7,200.00	$15	$108,000	
(2) excavation	CY	43,000.00	$15	$645,000	
(3) mesh	SF	36,000.00	$3.25	$117,000	
(4) backfill	CY	15,000.00	$5	$75,000	
b. D/S access road	LF	1,500.00	$80	$120,000	
(1) rockbolts	LF	1,100.00	$15	$16,500	
(2) excavation	CY	47,000.00	$5	$235,000	
(3) rock excavation	CY	9,000.00	$15	$135,000	
(4) mesh	SF	3,500.00	$3.25	$11,375	
(5) safety fence	LF	450.00	$25	$11,250	
					$1,834,125
5. Excavation (Intake)					
a. Overburden	CY	80,000.00	$5	$400,000	
b. Rock	CY	70,000.00	$15	$1,050,000	
c. Foundation Prep.	SY	9,700.00	$40	$388,000	
d. Slope Treatment					
(1) rockbolts	LF	21,000.00	$15	$315,000	
(2) shotcrete	CY	820.00	$300	$246,000	
(3) fencing	LF	900.00	$25	$22,500	
(4) consolidation grout	LS	1.00	$100,000	$100,000	
					$2,521,500
6. Excavation (Preformed Plunge Pool)					
a. Overburden	CY	92,000.00	$5	$460,000	
b. Foundation Backfill	CY	3,000.00	$5	$15,000	
b. Riprap	CY	2,000.00	$55	$110,000	
c. Slope Treatment					
(1) tiebacks	LF	7,000.00	$15	$105,000	
					$690,000
7. Excavation (D/S Control Sructure)					
a. Overburden	CY	105,000.00	$5	$525,000	
b. Rock	CY	37,000.00	$15	$555,000	
c. Foundation Prep.	SY	3,000.00	$40	$120,000	
d. Slope Treatment					
(1) rockbolts	LF	8,000.00	$15	$120,000	
(2) shotcrete	CY	280.00	$300	$84,000	
(3) mesh	SF	2,600.00	$3.25	$8,450	
(4) fencing	LF	800.00	$25	$20,000	
					$1,432,450

Santa Ana Project - Seven Oaks Outlet Works - Cost Estimate
Downstream Control - Steel RO Conduit - High Level Intake Tower

Item	Unit	Quantity	Unit Cost	Item Cost	Subtotal
8. Tunnel Excavation (25' Horseshoe)					
a. Excavation	CY	26,500.00	$100	$2,650,000	
b. Support					
(1) shotcrete	CY	1,800.00	$300	$540,000	
(2) rockbolts	LF(Tunnel)	1,620.00	$30	$48,600	
(3) drains	LS	1.00	$100,000	$100,000	
(4) ribs	LF(Tunnel)	1,620.00	$325	$526,500	
(5) grout ring	LS	1.00	$100,000	$100,000	
c. Floor Drain System	LS	1.00	$50,000	$50,000	
d. Contact Grouting	LS	1.00	$250,000	$250,000	
e. Gravel Drain	CY	2,500.00	$10	$25,000	
					$4,290,100
9. Instrumentation					
a. Geotechnical	LS	1.00	$100,000	$100,000	
b. Hydraulic	LS	1.00	$25,000	$25,000	
					$125,000
10. Concrete					
a. Intake Structure					
(1) Below el. 2156.0	CY	8,700.00	$200	$1,740,000	
(2) Above el. 2156.0	CY	4,600.00	$400	$1,840,000	
b. Access bridge	LS	1.00	$60,000	$60,000	
c. D/S Control Structure	CY	2,500.00	$225	$562,500	
d. Channel & Flip Bucket	CY	2,000.00	$225	$450,000	
e. Tunnel Floor & Walls	CY	6,500.00	$250	$1,625,000	
f. Conduit Support	CY	6,500.00	$150	975,000.00	
g. Cutoff Wall	CY	500.00	225.00	112,500.00	
					$7,365,000
11. Miscellaneous metals					
a. Handrails and ladders	LS	1.00	$60,000	$60,000	
b. Grating and hatches	LS	1.00	$50,000	$50,000	
					$110,000
12. Structural steel					
a. RO bulkhead & guides	LB	90,000.00	$3.30	$297,000	
b. Trashracks	LS	1.00	$20,000	$20,000	
c. RO Conduit & supports	LB	2,600,000.00	$2	$5,200,000	
d. Low Flow Pipes & Sup's	LS	1.00	$600,000	$600,000	
					$6,117,000

Santa Ana Project - Seven Oaks Outlet Works - Cost Estimate
Downstream Control - Steel RO Conduit - High Level Intake Tower

Item	Unit	Quantity	Unit Cost	Item Cost	Subtotal
13. Mechanical Equipment					
a. Floatwell mechanisms	LS	1.00	$30,000	$30,000	
b. Water supply	LS	1.00	$30,000	$30,000	
c. Structure Drains	LS	1.00	$15,000	$15,000	
d. RO gates, frames, cylinders & operators	LS	1.00	$1,500,000	$1,500,000	
e. Fuel tank & generator	LS	1.00	$70,000	$70,000	
f. Sanitary Facilities	LS	1.00	$10,000	$10,000	
g. Heating & ventilating	LS	1.00	$20,000	$20,000	
h. 2'x 3.5' gating	LS	1.00	$120,000	$120,000	
i. Reserved	LS	1.00	$0	$0	
j. U/S Sluice gates (3)	LS	1.00	$125,000	$125,000	
k. Reserved	LS	1.00	$0	$0	
l. Min. discharge gating	LS	1.00	$50,000	$50,000	
					$1,970,000
14. Electrical Equipment	LS	1.00	$250,000	$250,000	
					$250,000
15. Architectural Features	LS	1.00	$100,000	$100,000	
					$100,000
Sub-Total Const. costs		$29,113,175			$29,113,175
Contingency	15.0%	$4,366,976			
Total Construction Costs Downstream Control		$33,480,151			

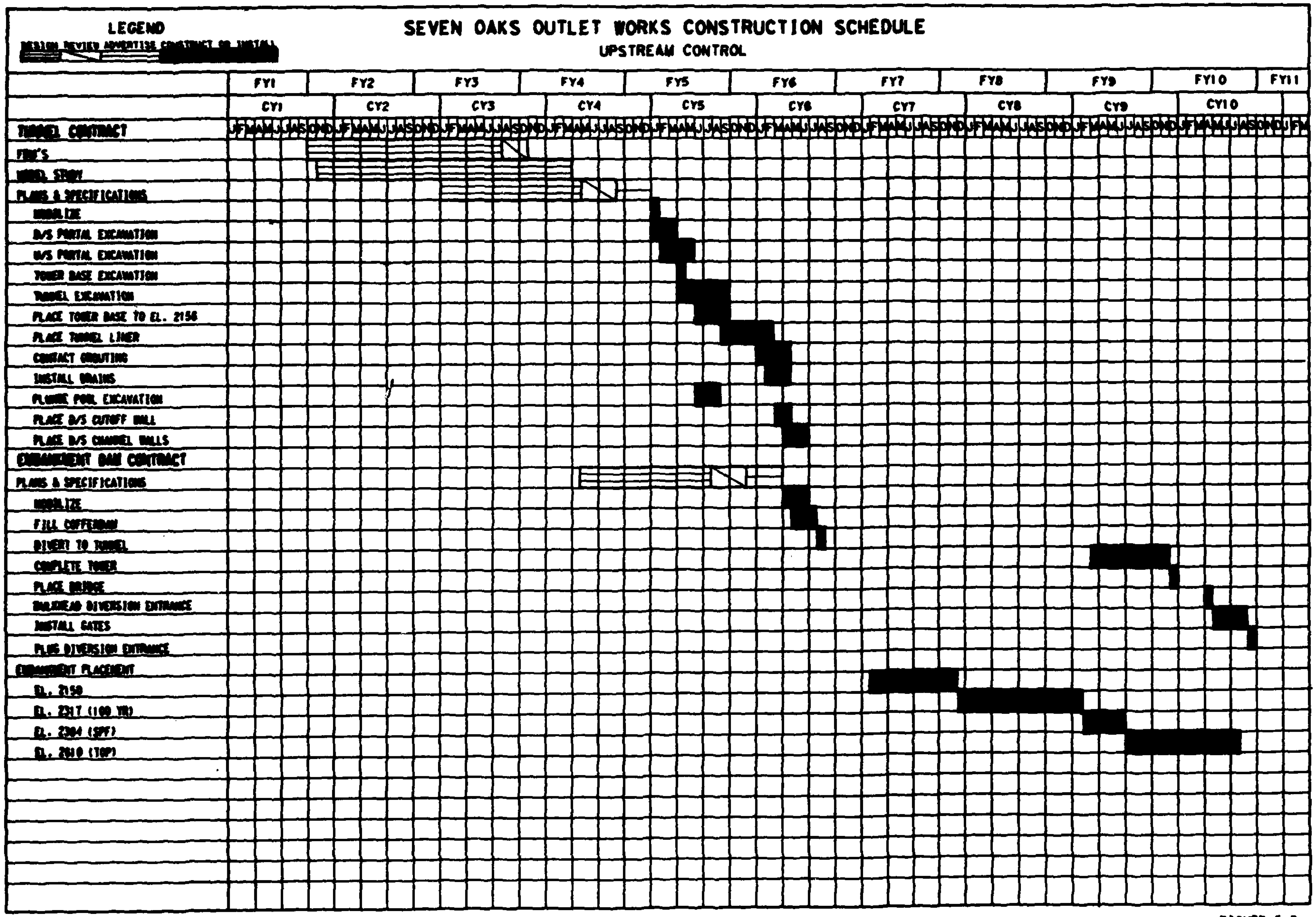

LEGEND
SEVEN OAKS OUTLET WORKS CONSTRUCTION SCHEDULE
UPSTREAM CONTROL
FY1
FY2
FY3
FY4
FY5
FY6
FY7
FY8
FY9
FY10
FY11
CY1
CY2
CY3
CY4
CY5
CY6
CY7
CY8
CY9
CY10
TUNNEL CONTRACT
FDM'S
MODEL STUDY
PLANS & SPECIFICATIONS
MOBILIZE
D/S PORTAL EXCAVATION
U/S PORTAL EXCAVATION
TOWER BASE EXCAVATION
TUNNEL EXCAVATION
PLACE TOWER BASE TO EL. 2156
PLACE TUNNEL LINER
CONTACT GROUTING
INSTALL DRAINS
PLUNGE POOL EXCAVATION
PLACE D/S CUTOFF WALL
PLACE D/S CHANNEL WALLS
EMBANKMENT DAM CONTRACT
PLANS & SPECIFICATIONS
MOBILIZE
FILL COFFERDAM
DIVERT TO TUNNEL
COMPLETE TOWER
PLACE BRIDGE
BULKHEAD DIVERSION ENTRANCE
INSTALL GATES
PLUG DIVERSION ENTRANCE
EMBANKMENT PLACEMENT
EL. 2150
EL. 2317 (100 YR)
EL. 2384 (SPF)
EL. 2610 (TOP)

FIGURE 5-3

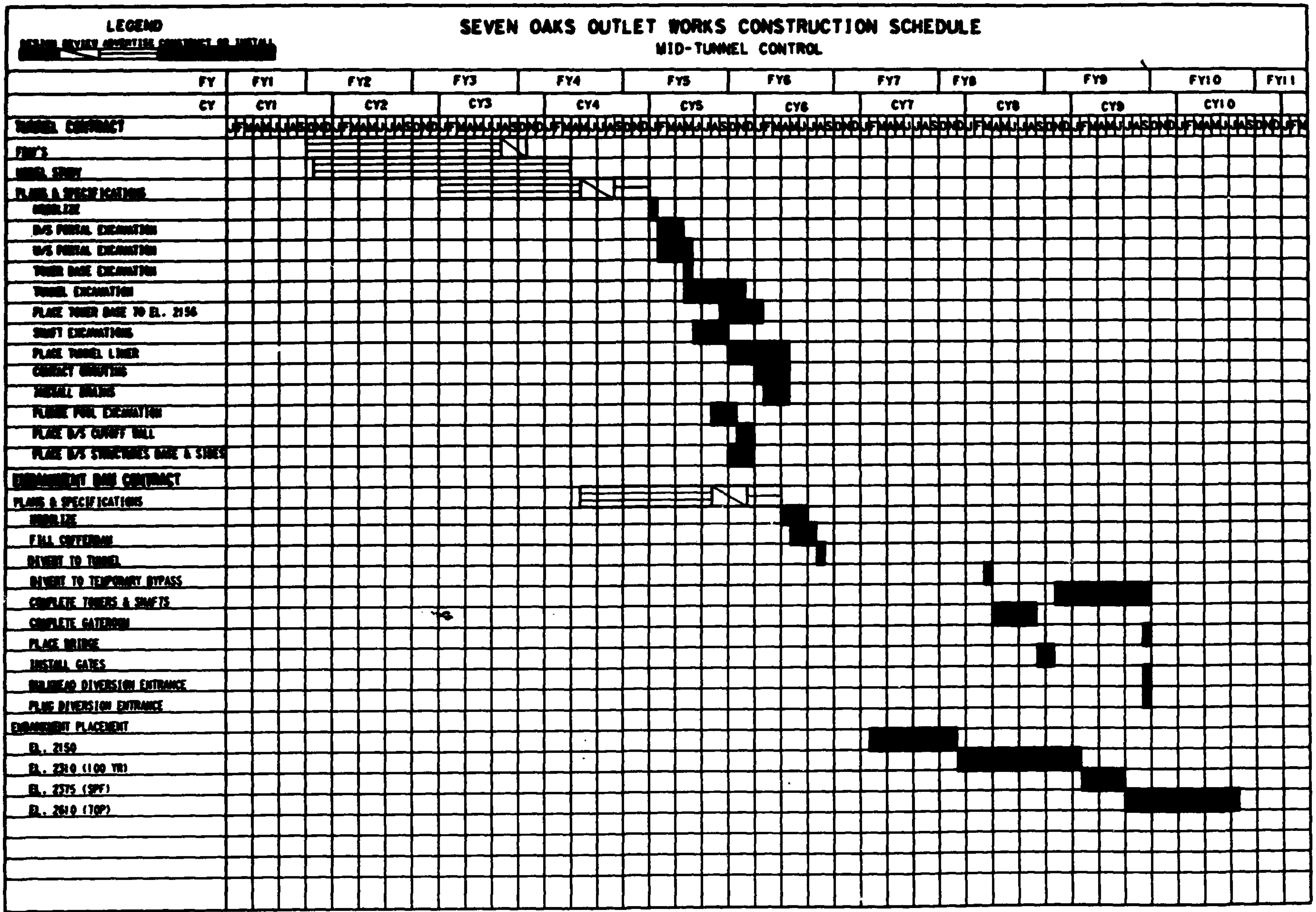
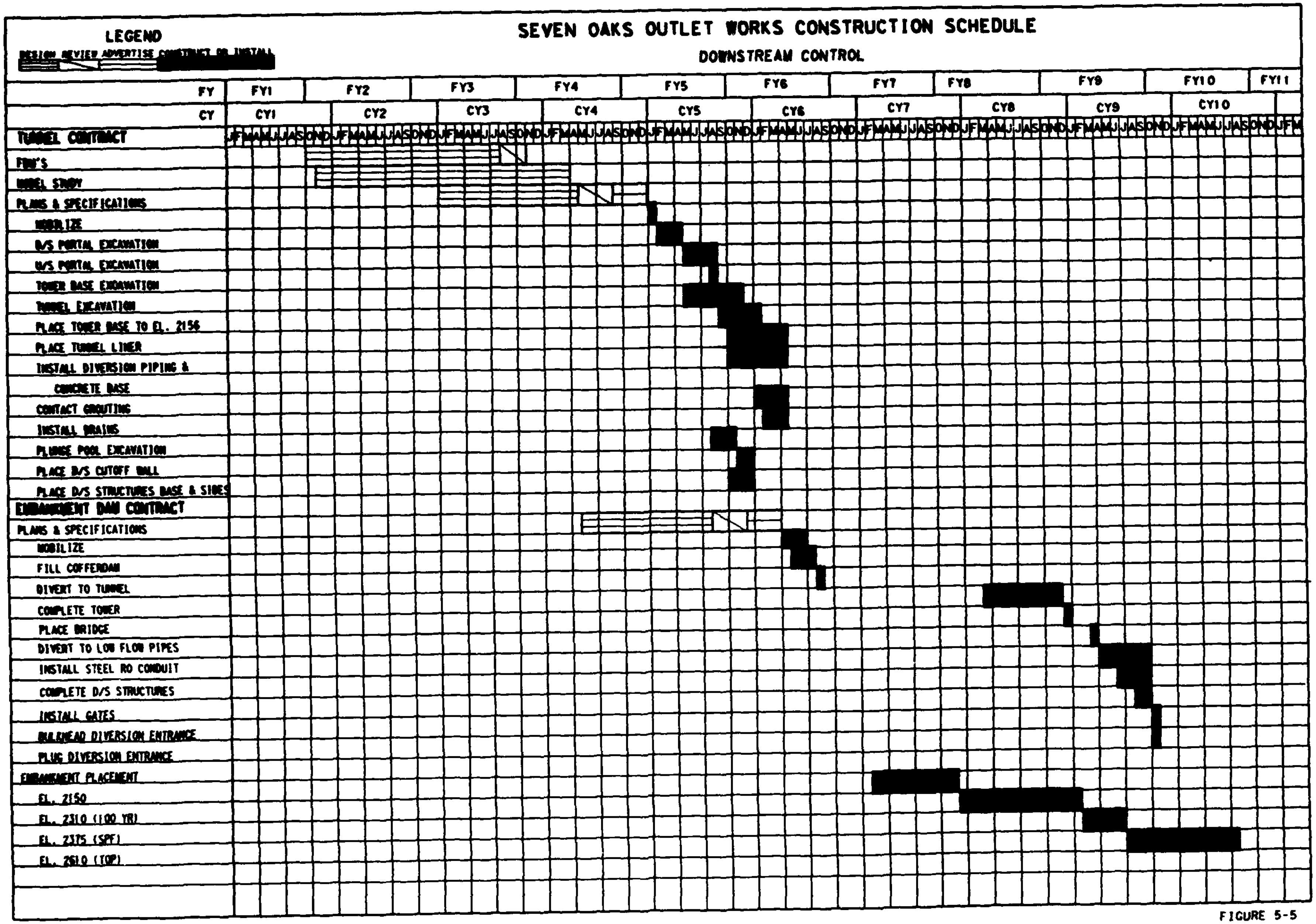

LEGEND
SEVEN OAKS OUTLET WORKS CONSTRUCTION SCHEDULE
MID-TUNNEL CONTROL
FY
CY
FY1
FY2
FY3
FY4
FY5
FY6
FY7
FY8
FY9
FY10
FY11
CY1
CY2
CY3
CY4
CY5
CY6
CY7
CY8
CY9
CY10
TUNNEL CONTRACT
PLANS & SPECIFICATIONS
D/S PORTAL EXCAVATION
U/S PORTAL EXCAVATION
TOWER BASE EXCAVATION
TUNNEL EXCAVATION
PLACE TOWER BASE TO EL. 2156
SHAFT EXCAVATIONS
PLACE TUNNEL LINER
PLUNGE POOL EXCAVATION
PLACE D/S CUTOFF WALL
PLACE D/S STRUCTURES BASE & SIDES
EMBANKMENT DAM CONTRACT
PLANS & SPECIFICATIONS
FILL COFFERDAM
DIVERT TO TUNNEL
DIVERT TO TEMPORARY BYPASS
COMPLETE TOWERS & SHAFTS
COMPLETE GATEROOM
PLACE BRIDGE
INSTALL GATES
BULKHEAD DIVERSION ENTRANCE
PLUG DIVERSION ENTRANCE
EMBANKMENT PLACEMENT
EL. 2150
EL. 2310 (100 YR)
EL. 2375 (SPF)
EL. 2610 (TOP)

FIGURE 5-4



LEGEND
DESIGN REVIEW ADVERTISE CONSTRUCT OR INSTALL
SEVEN OAKS OUTLET WORKS CONSTRUCTION SCHEDULE
DOWNSTREAM CONTROL
FY
CY
FY1 FY2 FY3 FY4 FY5 FY6 FY7 FY8 FY9 FY10 FY11
CY1 CY2 CY3 CY4 CY5 CY6 CY7 CY8 CY9 CY10
TUNNEL CONTRACT
FDM'S
MODEL STUDY
PLANS & SPECIFICATIONS
MOBILIZE
D/S PORTAL EXCAVATION
U/S PORTAL EXCAVATION
TOWER BASE EXCAVATION
TUNNEL EXCAVATION
PLACE TOWER BASE TO EL. 2156
PLACE TUNNEL LINER
INSTALL DIVERSION PIPING & CONCRETE BASE
CONTACT GROUTING
INSTALL DRAINS
PLUNGE POOL EXCAVATION
PLACE D/S CUTOFF WALL
PLACE D/S STRUCTURES BASE & SIDES
EMBANKMENT DAM CONTRACT
PLANS & SPECIFICATIONS
MOBILIZE
FILL COFFERDAM
DIVERT TO TUNNEL
COMPLETE TOWER
PLACE BRIDGE
DIVERT TO LOW FLOW PIPES
INSTALL STEEL RO CONDUIT
COMPLETE D/S STRUCTURES
INSTALL GATES
BULKHEAD DIVERSION ENTRANCE
PLUG DIVERSION ENTRANCE
EMBANKMENT PLACEMENT
EL. 2150
EL. 2310 (100 YR)
EL. 2375 (SPF)
EL. 2610 (TOP)

FIGURE 5-5

SECTION 6

HYDRAULIC DESIGN

6.1 General.

a. Hydraulic Design. The hydraulic design of the Seven Oaks Dam outlet works is based upon the required project releases listed in the operation schedule. The hydraulic design of this project conforms to the usual procedures for structures of this type, as outlined in engineering manuals, hydraulic design criteria, and based on the results of model and prototype studies.

b. Design Alternatives. Three of the four alternatives (described in paragraph 1.1) considered for RO control have been designed: upstream control with a horizontal gallery; downstream control with a pressurized conduit; and mid-tunnel control with a vertical shaft.

(1) System Description. The RO works consists of two intake systems and three separate control systems which will regulate flow. Two intake systems, a high level intake with a 36-foot-diameter wet well and a low pool multilevel withdrawal intake with an 8-foot by 8.5-foot wet well, will be used. This is necessary to prevent sediment from passing through the outlet works and to avoid dead storage during the 100-year design life of the project. A passageway connects the two drop wells and can be closed with a gate (wet well sluice gate). Flow can pass either direction between wet wells. A trashrack with 6-inch by 6-inch openings will be provided in the passageway between drop wells. This will prevent material which is large enough to pass through the high level intake from jamming the minimum discharge line gate. The multilevel withdrawal intake is required so the system can pass flows at low pool elevations. The high level intake will be used for high pool elevations and when the expected sediment deposition in the forebay rises to a level where the low level intake can no longer be used. The multilevel withdrawal intake consists of multiple levels of ports that can be stoplogged as the sediment level rises. Three separate control systems

will be used: minimum discharge line, low flow bypass, and main RO. The three systems are necessary to accurately regulate flow over the wide range of pool elevations and design discharges. The minimum discharge line, the low flow bypass, and the main RO outlet are designed to pass flows ranging from approximately 10 to 90 cfs, 50 to 600 cfs, and 170 to 8,000 cfs, respectively (values vary for each alternative). The overlap in discharge for the different systems allows for some flexibility in operation. The low flow bypass and main RO conduit tie into the wet well for the high level intake. The minimum discharge line entrance is located at the base of the wet well for the multilevel withdrawal intake. By closing or opening the gate in the passageway and/or by using one or more of the bulkheads provided at the upstream end of each conduit, any conduit or combination of conduits may be operated while the remaining conduits are repaired or inspected (with the exception of mid-tunnel control). There are combinations which are not recommended for normal operation; these will be discussed later in the text. At low pool elevations it is also possible to dewater the wet well for the high level intake while passing flow through the minimum discharge line. For the downstream control option, the minimum discharge line and low flow bypass will be used for summer diversion. Both lines are required for diversion capacity while the 11-foot-diameter pressure conduit is being installed. Brief descriptions of each alternative follow.

(2) Upstream Control. The main RO gates, low flow bypass gate, and minimum discharge gate are located within a single structure located at the base of the intake tower. All regulated discharge flows into an 18-foot-diameter, 32-foot-high oblong tunnel in which the lower 17 feet are used for open channel flow. The upper 15 feet will be used for downstream access to the gates and air supply passages. Flow will exit the RO conduit approximately 1,605 feet downstream of the gates. An alternate design for the minimum discharge line is a 3-foot-diameter pressure pipe originating at the bottom of the multilevel withdrawal well that will carry flow the length of the RO works and will be regulated at the downstream end by a cone valve. All design computations for upstream control were based on initial design criteria. Some of the initial design criteria has changed as discussed in paragraph 1.5. The analysis will be reworked using the current criteria and operation schedules if the upstream control alternative is selected for the feature design.

(3) Downstream Control. Flow is regulated by control gates located approximately 1,680 feet downstream of the intake tower. An 11-foot-diameter steel pressure conduit will carry flow from the intake to the main RO gates. The gates will be accessed from downstream of the dam. For low flow and minimum discharges, 3.5-foot- and 3.25-foot-diameter steel pipes, respectively, will carry flow under pressure for a distance of about 1,680 feet to the low flow and minimum discharge gates. Flow from each gate will discharge into its own exit chute.

(4) Mid-Tunnel Control. Flow is regulated by control gates located approximately 450 feet downstream of the intake tower. Access to the gates is from the top of the dam through the 520-foot-high mid-tunnel control shaft. An 18-foot-diameter horseshoe conduit will pass flow under pressure between the intake and the low flow and main RO gates. A 2-foot-diameter concrete pipe will carry minimum discharges under pressure from the multilevel withdrawal intake tower to the mid-tunnel control area. Downstream of the RO gates, all discharge will be open channel flow for 1,067 feet through an 18-foot-diameter horseshoe shaped tunnel. An alternate design for the minimum discharge line is a 3-foot-diameter pressure pipe originating at the bottom of the multilevel withdrawal well that will carry flow the length of the RO works and will be regulated at the downstream end by a cone valve.

c. Aeration. Due to the high velocities downstream of control gates, special measures have been incorporated for each alternative into the outlet works in the form of aeration offsets to prevent incipient cavitation.

d. Model Study. Whichever alternative is selected for the Seven Oaks Dam outlet works should be model tested for the following reasons: high flow velocities and corresponding cavitation potential; difficulty in accurately defining flow profiles associated with offsets; to evaluate the combining of high velocity jets from the gates for upstream and mid-tunnel control; and to evaluate hydraulic loading on main RO gates due to the downstream and mid-tunnel control transitions.

e. Instrumentation. The design of hydraulic prototype instrumentation will be finalized in the feature design. Coordination has been initiated with the prototype instrumentation section at the Waterways Experiment Station. This will allow the recommended instrumentation to be incorporated into the design early in the feature level.

6.2 Trash Structure.

a. General. The trash structure (see plates 2-3, 3-3, 3-4, 4-3, and 4-4) for the three design alternatives is designed to provide minimum resistance to flow and to pass all material except that which would make the outlet inoperative. Because of the large quantity of trash expected and due to the inaccessibility of the trash deck at pool elevations above 2,299, the net area of trash struts is larger than the minimum recommended by general guidance in EM 1110-2-1602. The trash structure consists of 18-inch upright concrete beams supported by 18-inch horizontal struts. The openings are 3.33 feet by 3.33 feet. The top of trash structure opening is at El. 2,292.5 and the bottom is at El. 2,265. There are 116 openings providing a net area for the trash structure equal to 1,289 square feet (except for mid-tunnel control which has 122 openings). The gross area of the trash structure is 2,308 square feet. An average velocity of 6.2 feet per second (fps) will occur through the trash struts at the design discharge of 8,000 cfs. Guidance from EM 1110-2-1602 recommends that velocities through the trash struts not exceed 15 fps. Flow will enter the trash structure fairly uniformly because of elevated invert and circular design of the high level intake. Therefore, local net-area velocities should not deviate much from average velocities. Since low velocities are expected through trash struts and flow should be approximately uniform, a flow net analysis was not necessary.

b. Trash Strut Energy Losses. Equation 11, page 366, "Design of Small Dams," Bureau of Reclamation, was used to determine energy losses through trash struts. Trash struts were assumed 50 percent clogged for capacity design computations. For this condition a loss coefficient K value of .025 was calculated. Loss coefficients determined for velocity and gate rating computations were lower than the value of 0.02 recommended

as general guidance in EM 1110-2-1602. This is because the net area of trash struts is larger than the minimum required. The loss coefficients were used with average velocity heads in the gate passages, just upstream of the gates.

6.3 Intake Tower.

a. General. Since large sediment loads are expected, a high level intake will be used for the three design alternatives (see plates 2-3, 3-3, and 4-3). The intake tower sill is at El. 2,265, 165 feet above the intake conduit invert. The height is based on 165 feet of predicted sediment deposition over 100 years. Flow passing through the struts enters into a 36-foot-diameter wet well. The maximum average velocity through the wet well will be 7.9 fps at the design discharge of 8,000 cfs. The maximum discharge operating criteria is shown on table 6-1 and figure 6-1 for present conditions and table 6-2 and figure 6-2 for future conditions (note this is most recent operating criteria, original operating criteria was modified). Vortex computations are based on guidance in EM 1110-2-1602, plate C-35. A graph of allowable discharge versus pool elevation for each alternative is shown on figures 6-3, 6-4, and 6-5. Vortices should not form for operating conditions above pool El. 2,265. Due to rising and falling pools below El. 2,265, vortices may develop as ports are submerged and exposed. Vortices are not expected to be a problem, but this will be further investigated at the FDM level. For a rising pool, discharges from pool El. 2,265 through pool El. 2,298 should be made using the minimum discharge line (see paragraphs on minimum discharge line, paragraphs 6.5 through 6.7) to prevent trash from being drawn directly into the trash struts. This will be accomplished by closing the wet well sluice gate between the low pool multilevel withdrawal well (see paragraph 6.4) and the main well. The manual control for the wet well sluice gate will be located on the trash deck. For a falling pool, between pool El. 2,298 and pool El. 2,265, discharges can be made through the main RO gates or the low flow line. Below pool El. 2,265 the wet well sluice gate is to be opened and provisions will be made to clean trash struts before the next flood event.

Table 6-1. Seven Oaks Dam Operation Schedule

Initial Conditions*

Elevation (Feet)	Storage (Acre-Feet)	Outflow (CFS) Minimum	Rising**	Falling**	Maximum
2,100	0	0	0	0	0
2,110	18	10	0	10****	20
2,150	552	10	0	500****	500
2,200	2,968	10	500	500	500
2,264	10,120	10	500	500	500
2,265	10,270	10	50	500	500***
2,269	10,882	10	50	1,000	1,000***
2,273	11,512	10	50	1,500	1,500***
2,278	12,324	10	50	2,000	2,000***
2,298	15,906	10	50	2,000	2,000***
2,299	16,099	10	500	2,000	2,000
2,300	16,293	100	500	2,030	5,000
2,400	43,327	200	500	4,340	6,500
2,500	90,398	200	500	6,560	7,000
2,570	137,830	200	500	6,950	7,800
2,580	145,608	200	500	7,000	8,000
(Spillway Crest)					
2,585	149,604	0	0	0	0
2,590	153,673	0	0	0	0
2,600	162,032	0	0	0	0
2,610 (Top of Dam)	170,685	0	0	0	0

FOOTNOTES:

*After dam construction, initial operation.

**Rising pool operation used until flood event at Prado Dam has passed; falling pool operation is then implemented.

***For rising pool, maximum discharge should be limited to 50 cfs to prevent floating debris from accumulating on trash structure.

****Release is equal to 10 to 20 cfs plus inflow up to El. 2,200 feet in order to drain debris pool; when debris pool is required to be maintained, outflow equals inflow as long as it does not exceed the maximum.

Table 6-2. Seven Oaks Dam Operation Schedule

Future Conditions*

Elevation (Feet)	Storage (Acre-Feet)	Outflow (CFS) Minimum	Rising**	Falling**	Maximum
2,265	0	0	0	0	0
2,269	10	10	50	****	1,000***
2,273	38	10	50	****	1,500***
2,278	102	10	50	****	2,000***
2,298	758	10	50	****	2,000***
2,299	808	10	500	****	2,000
2,300	859	100	500	500	2,000
2,325	2,773	100	500	2,000	4,900
2,350	5,917	100	500	2,075	5,500
2,400	16,450	200	500	2,840	6,000
2,450	33,985	200	500	4,000	6,400
2,500	58,858	200	500	5,700	6,750
2,525	74,061	200	500	6,440	6,950
2,550	91,054	200	500	6,680	7,300
2,575	109,685	200	500	6,925	7,700
2,580 (Spillway Crest)	113,608	200	500	7,000	8,000
2,585	117,604	0	0	0	0
2,590	121,673	0	0	0	0
2,600	130,032	0	0	0	0
2,610 (Top of Dam)	138,685	0	0	0	0

FOOTNOTES:

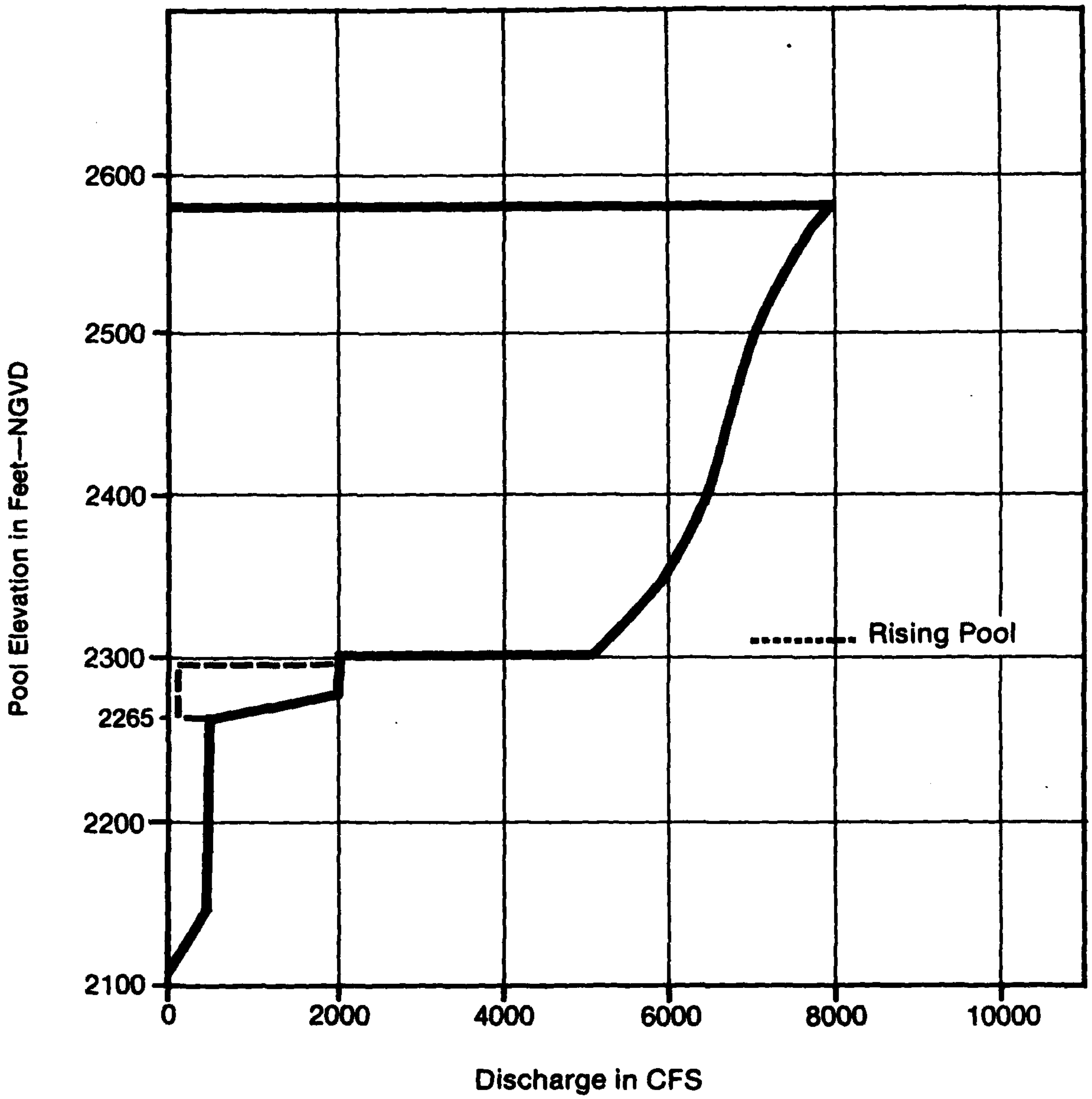
*Assuming 165 feet of sediment deposition above invert El. 2,100.

**Rising pool operation used until flood event at Prado Dam has passed; falling pool operation is then implemented.

***For rising pool, maximum discharge should be limited to 50 cfs to prevent floating debris from accumulating on trash structure.

****Release is equal to 10 to 20 cfs plus inflow up to El. 2,200 feet in order to drain debris pool; when debris pool is required to be maintained, outflow equals inflow as long as it does not exceed the maximum.

SEVEN OAKS
Maximum Operation Schedule
Initial Conditions

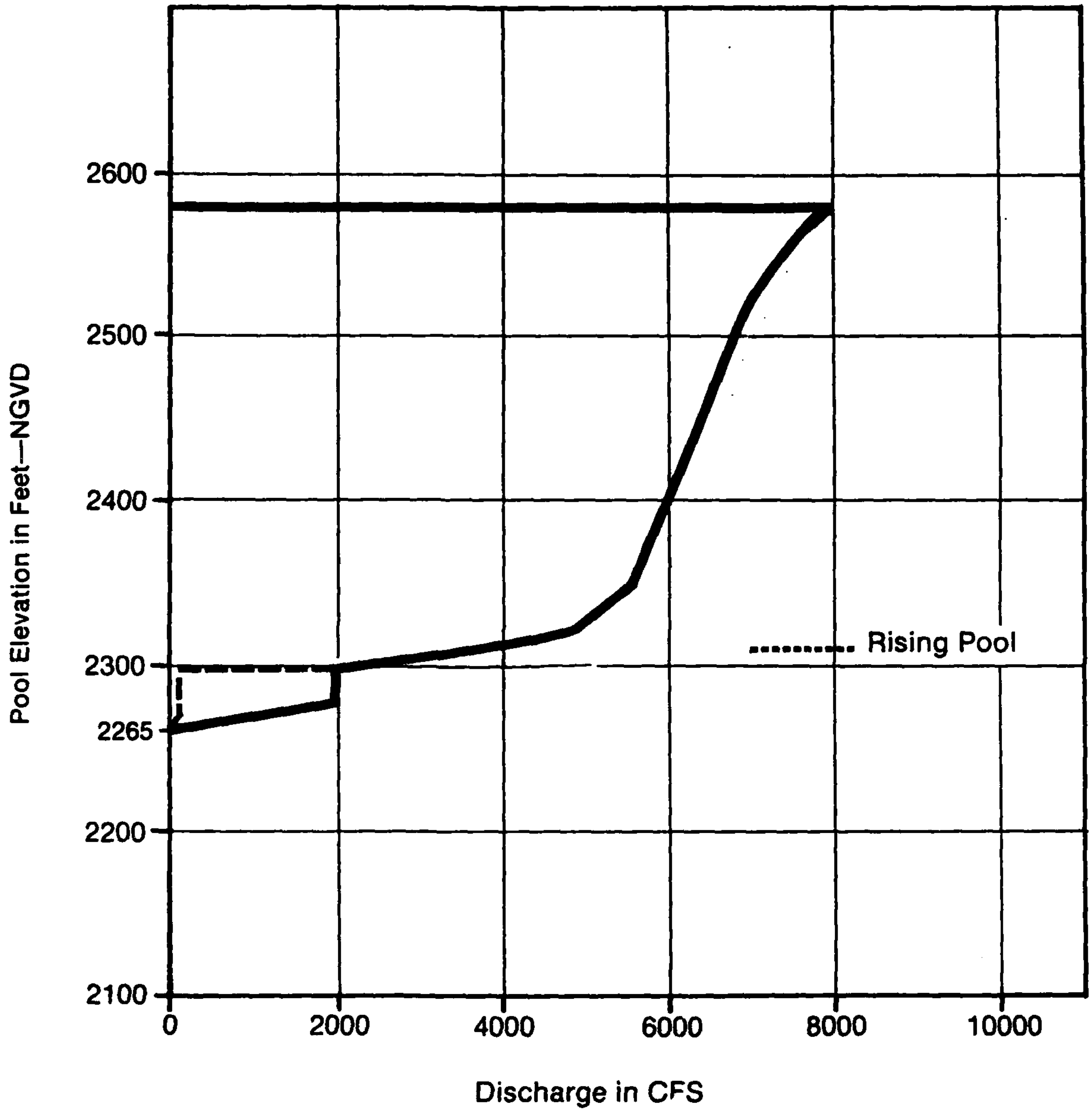


Note

1. Maximum discharge operating schedule.
2. For rising pools the minimum discharge line should be operated between pool elevations 2265 to 2298 at a maximum discharge of 50 cfs. This will prevent floating trash from accumulating on trash structure.

Figure 6-1 Maximum discharge operating conditions, initial conditions.

SEVEN OAKS
Maximum Operation Schedule
Future Conditions



Note

1. Maximum discharge operating schedule.
2. For rising pools the minimum discharge line should be operated between pool elevations 2265 to 2298 at a maximum discharge of 50 cfs. This will prevent floating trash from accumulating on trash structure.

Figure 6-2 Maximum discharge operating conditions, future conditions.

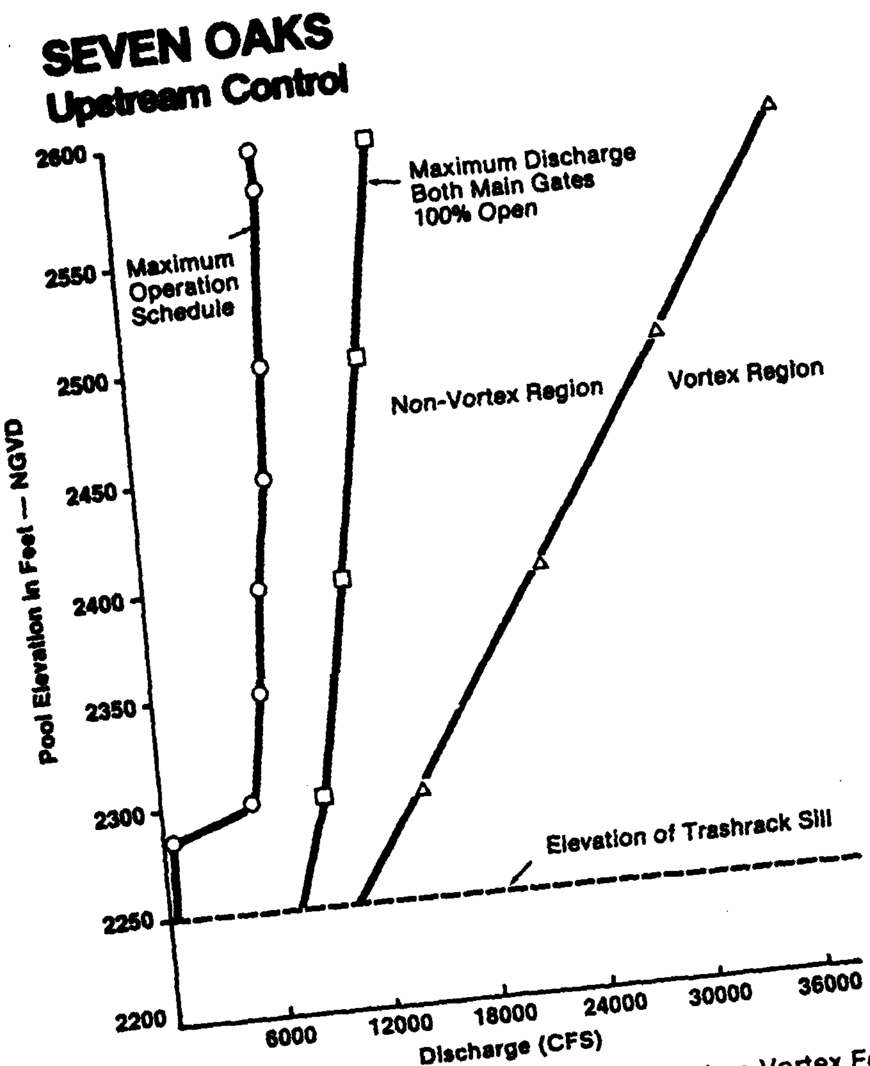


Figure 6-3 Maximum Acceptable Discharge Based on Vortex Formation

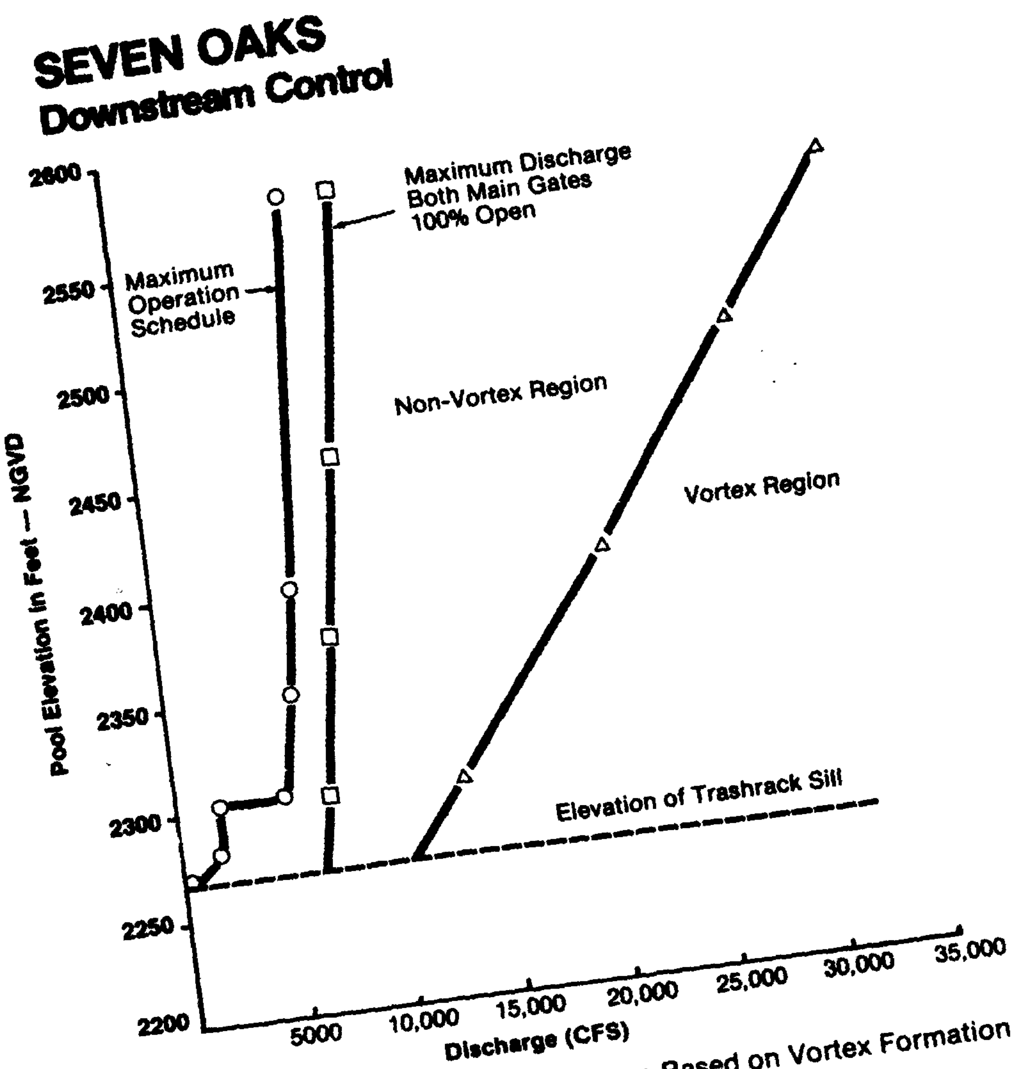


Figure 6-4 Maximum Acceptable Discharge Based on Vortex Formation

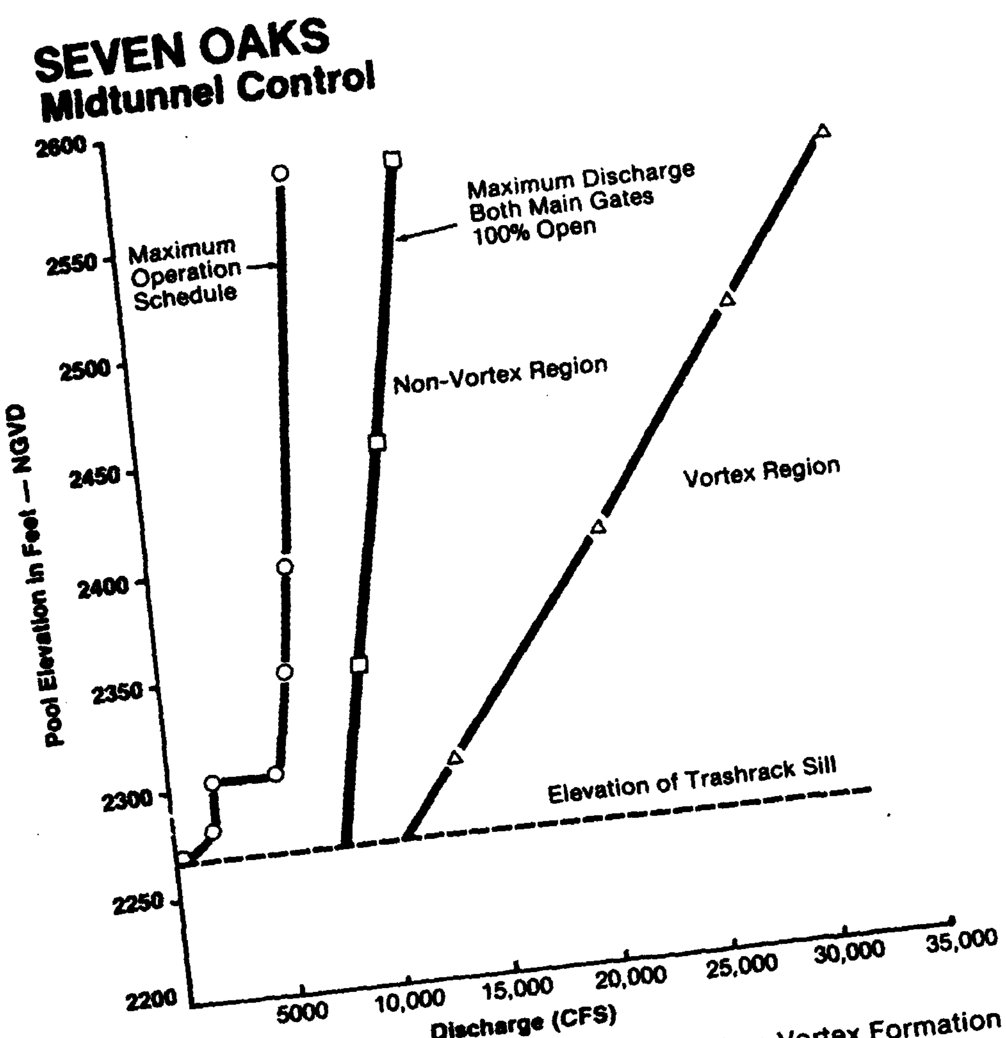


Figure 6-5 Maximum Acceptable Discharge Based on Vortex Formation

b. Intake Tower Energy Losses. Energy losses at entrance to high level intake were found to be negligible due to large area and corresponding low velocities. Friction losses in the wet well were estimated using Manning's "n" values of .008, .012, and .015 for velocity design, gate rating curves, and capacity design, respectively. Energy loss in the well due to friction was found to be negligible for all cases.

6.4 Multilevel Withdrawal System. To prevent dead storage, a multilevel withdrawal system is required in conjunction with the high level intake (see plates 2-3, 2-4, 3-4, 4-4). Seventeen rows of 2.25-foot-diameter ports, two ports per row, will be spaced at 10-foot intervals starting at El. 2,100. Ports will be covered with trashracks. As deposited sediment approaches the same elevation as a row of ports, that row of ports will be permanently closed with a stoplog. This will prevent sediment from entering RO works. Flow through the ports enters an 8.0- by 8.5-foot multilevel wet well. Flow out of the multilevel wet well is controlled by a 5-foot-wide by 7-foot-high wet well sluice gate which will be mechanically operated from the trash deck. This gate will only be used in either the fully open or closed position. When the wet well sluice gate is closed, flow can be passed through the minimum discharge line (see plates 2-3, 3-3, 4-3). When this gate is open, flow passes into the main (36-foot-diameter) wet well. Discharge rate will be controlled by one of the main operating gates (see paragraphs 6.5 through 6.7 on RO gate rating) or with the low flow bypass system (see paragraphs 6.5 through 6.7 on low flow). Approximately 50 feet of head above the channel bottom is required to pass 500 cfs and maintain control over flow. Below this, control may shift from RO gates due to head loss at ports and wet well sluice gates. With less head (fewer ports submerged), discharge will need to be less than 500 cfs. This matter will be addressed at the FDM level.

6.5 Upstream Control.

a. General. The upstream control alternative was chosen first for analysis for various construction and hydraulic reasons as discussed in paragraph 1.2.e.

Some of the initial design criteria has changed as discussed in paragraph 1.5. The change will not have a significant impact on the system operation, therefore the system was not reanalyzed. The text, tables, and figures in this section do not reflect the changes in criteria. The changes in criteria are: maximum design pool was lowered from El. 2,598 to El. 2,580; the high level intake sill was raised from El. 2,250 to El. 2,265 due to a higher expected sediment deposition level; and the maximum and minimum operation schedules have been altered. The operation schedules under which the upstream control alternate were analyzed are shown on tables 6-3 and 6-4, and figures 6-6 and 6-7. The current operation schedules are shown on tables 6-1 and 6-2 and figures 6-1 and 6-2. The analysis will be reworked using the current criteria if the upstream control alternative is selected for the feature design.

b. Regulating Outlet Intake. The RO entrance invert will be at El. 2,100 feet. Two RO intake passages, each 5 feet wide by 9 feet high, will be used. Each intake passage is sized to pass approximately 85 percent of design discharge, 8,000 cfs at pool El. 2,598 feet. Flow will be controlled with vertical slide gates. An upstream emergency gate will be included in each intake passage. Provisions have been made for maintenance bulkheading. Geometry of entrance curves is based on the design of Lost Creek Dam. The RO works at Lost Creek were model studied, as outlined in Technical Report No. 140-1, "Outlet Works for Lost Creek Dam, Rogue River, Oregon," Division Hydraulic Laboratory, U.S. Army Corps of Engineers, North Pacific Division. The Lost Creek project has operated successfully since 1977. Lost Creek has similar vertical entrance conditions and the model study indicated there would be no adverse pressures on the system. A compound ellipse has been selected for the roof curve, approximately a two on three ellipse for the upstream portion, and approximately a one on three ellipse for the downstream portion. This curve is designated type 5 for entrances flared in three directions in Technical Memorandum No. 2-428, Report No. 2, "Investigation of Entrances Flared in Three Directions and in One Direction," Waterways Experiment Station. The intake curve has been extended 9 feet into the wet well to provide an acceptable transition from the circular well to a flat front entrance. The side curves are conventional one on three ellipses. Horizontal and vertical entrance curves are based on width and height, respectively, of RO intake conduits.

Table 6-3. Seven Oaks Dam Operation Schedule*

Present Conditions (Upstream Control)

Elevation (Feet)	Storage (Acre-Feet)	Outflow (CFS) Minimum	Rising**	Falling**	Maximum
2,100	0	0	0	0	0
2,110	18	10	0	0	10
2,150		10	0	500	500
2,200	2,468	10	0	500	500
2,201	3,042	10	500	500	500
2,249	7,997	10	500	500	500
2,250	8,130	10	50	500	500***
2,283	13,170	10	50	500	500***
2,284	13,343	10	500	2,000	4,000
2,300	16,293	10	500	4,500	5,000
2,350	27,862	10	500	5,000	6,000
2,400	43,327	200	500	5,300	6,500
2,500	90,398	200	500	6,500	7,000
2,550	123,028	200	500	6,750	7,500
2,598	160,000	200	500	7,000	8,000
(Spillway Crest)					
2,600	162,032	0	0	0	0
2,610	170,685	0	0	0	0
2,620	179,634	0	0	0	0
2,630	188,880	0	0	0	0
(Top of Dam)					

FOOTNOTES:

*This operation schedule was used for analysis of the upstream control alternative. Any further analysis should be made using table 6-1.

**Rising pool operation used until flood event at Prado Dam has passed; then falling pool operation implemented.

***For rising pool, maximum discharge should be limited to 50 cfs to prevent floating debris from accumulating on trash structure.

Table 6-4. Seven Oaks Dam Operation Schedule*

Future Conditions (Upstream Control)**

Elevation (Feet)	Storage (Acre-Feet)	Outflow (CFS) Minimum	Rising***	Falling***	Maximum
2,100	0	0	0	0	0
2,110	0	0	0	0	0
2,200	0	0	0	0	0
2,249	0	0	0	0	0
2,250	0	0	0	0	0
2,283	560	10	0	500	500****
2,284	598	10	0	1,000	4,000
2,300	1,225	10	0	2,000	5,000
2,301	1,307	100	500	2,000	5,000
2,350	6,498	100	500	4,800	6,000
2,400	17,044	200	500	5,300	6,500
2,500	59,083	200	500	6,120	7,000
2,550	91,100	200	500	6,500	7,500
2,598	128,000	200	500	7,000	8,000
(Spillway Crest)					
2,600	130,000	0	0	0	0
2,610	138,685	0	0	0	0
2,620	147,634	0	0	0	0
2,630	156,880	0	0	0	0
(Top of Dam)					

FOOTNOTES:

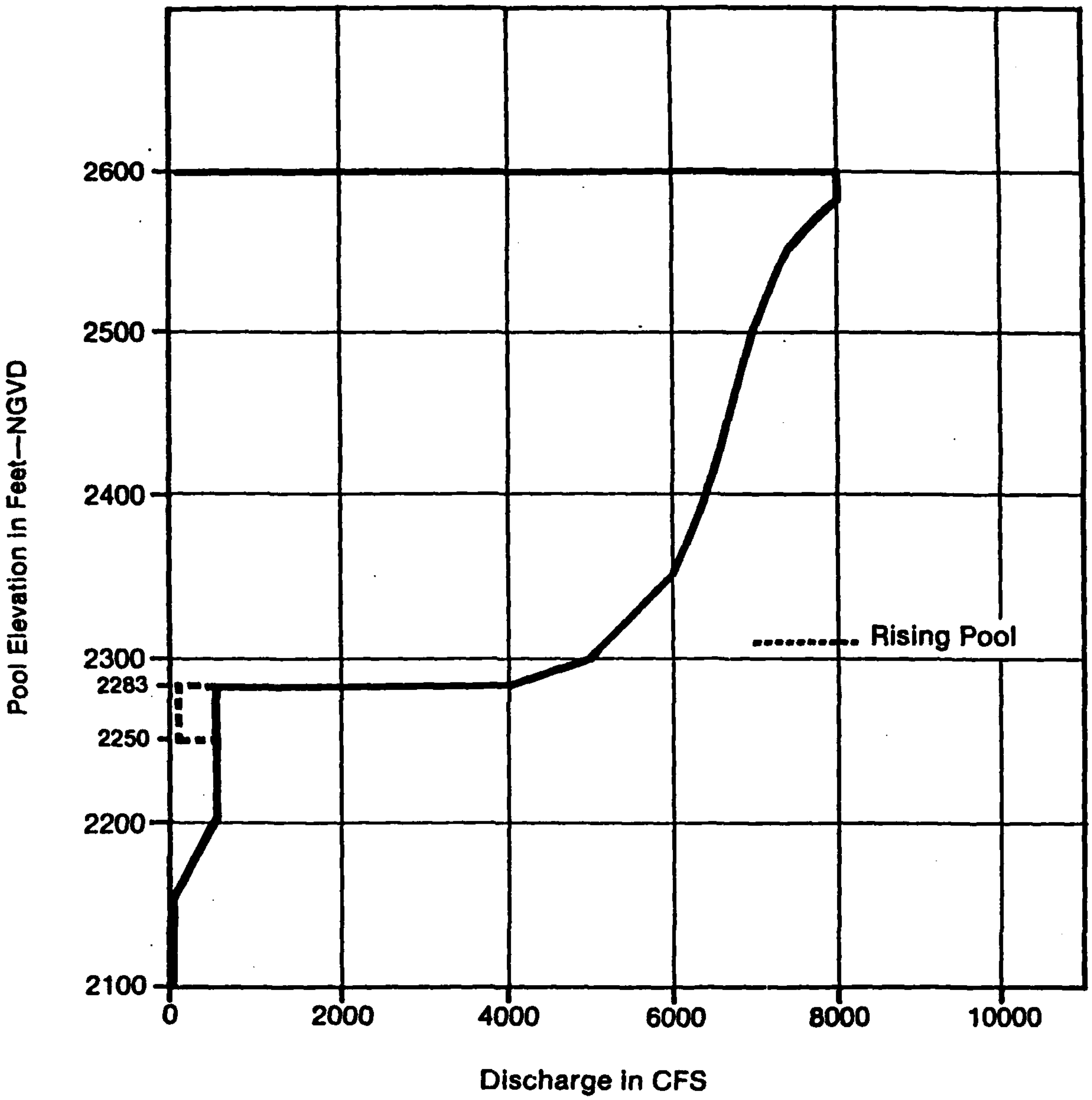
*This operation schedule was used for analysis of the upstream control alternative. Any further analysis should be made using table 6-2.

**Assuming 150 feet of sediment deposition above invert El. 2,100.

***Rising pool operation used until flood event at Prado Dam has passed; then falling pool operation implemented.

****For rising pool, maximum discharge should be limited to 50 cfs to prevent floating debris from accumulating on trash structure.

SEVEN OAKS
Maximum Operation Schedule
Present Conditions (Upstream Control)

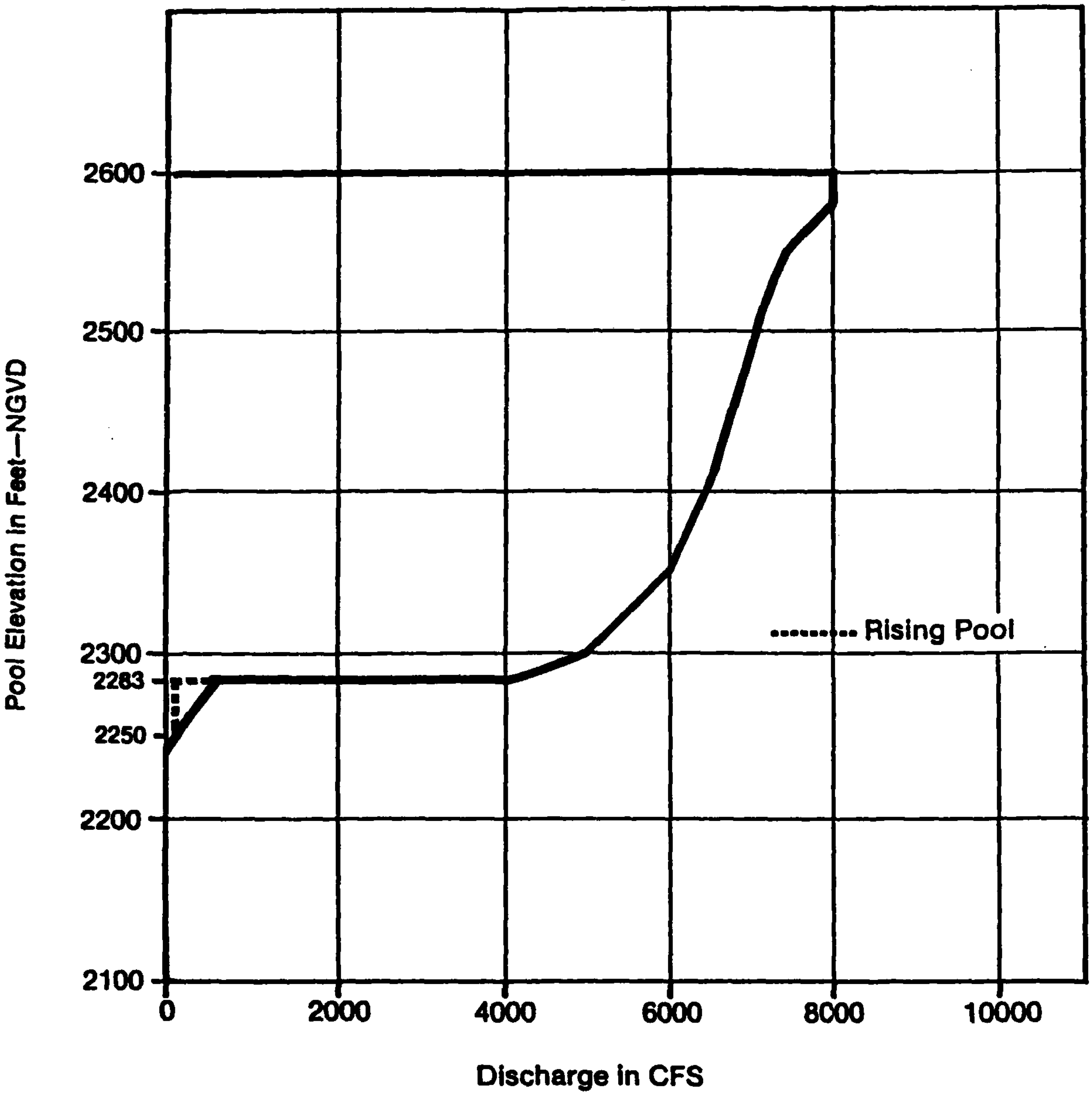


Note

1. Maximum discharge operating schedule.
2. For rising pools the minimum discharge line should be operated between pool elevations 2250 to 2283 at a maximum discharge of 50 cfs. This will prevent floating trash from accumulating on trash structure.
3. This operation schedule was used f(analysis of the Upstream Control alternativ Any further analysis should be made usir Fig. 6-1

Figure 6-6 Maximum discharge operating conditions, present conditions.

SEVEN OAKS
Maximum Operation Schedule
Future Conditions (Upstream Control)



Note

1. Maximum discharge operating schedule.
2. *For rising pools the minimum discharge line should be operated between pool elevations 2250 to 2283 at a maximum discharge of 50 cfs. This will prevent floating trash from accumulating on trash structure.*
3. This operation schedule was used for analysis of the Upstream Control alternative. Any further analysis should be made using Fig. 6-2

Figure 6-7 Maximum discharge operating conditions, future conditions.

c. Regulating Outlet Gate Rating.

(1) General. The RO rating curves for various gate openings are shown on figure 6-8 (high level intake) and figure 6-9 (multilevel withdrawal system). Figure 6-10 shows regions where minimum discharge line, low flow bypass, and main regulating outlet are required when multilevel withdrawal system is used. Average values of loss coefficients have been used to approximate actual operating conditions. Minimum, average, and maximum loss coefficients have been calculated from the pool to the regulating gates and were used for velocity, rating curve, and capacity design, respectively.

(2) Energy Loss Coefficients for High Level Intake. Trash strut coefficients were determined with 0 percent, 25 percent, and 50 percent clogging, using equation 11, page 366, "Design of Small Dams," Bureau of Reclamation. Friction loss coefficients, K values, in the drop well were calculated based on the Darcy-Weisbach equation using equivalent Mannings "n" values of .008, .012, and .015. Energy loss in the well due to friction was found to be negligible. The RO entrance loss coefficients used were .10, .15, and .20. References are EM 1110-2-1602, Lost Creek computations, and "Design of Small Dams," Bureau of Reclamation. Manning's "n" values used to calculate friction losses in the tunnel upstream of the gates are .008, .012, and .015. The total minimum, average, and maximum loss coefficients between the pool and the section just upstream of the RO gate, relative to the area of each main intake conduit cross section, are .158, .266, and .381, respectively. Values are summarized in table 6-5. The total loss coefficients are higher than the .16 for capacity and .10 for velocity suggested in EM 1110-2-1602. The additional energy loss is due to the drop well and the reentrance condition into the intake. The intake design at Seven Oaks is similar to that used for Lost Creek Dam, a design that was model studied and has proven successful during more than 10 years of prototype performance. Therefore, intake loss coefficients were based on this project. Also, loss coefficients were separated into components since there is no "total intake system" identical to that proposed at Seven Oaks.

SEVEN OAKS
Upstream Control

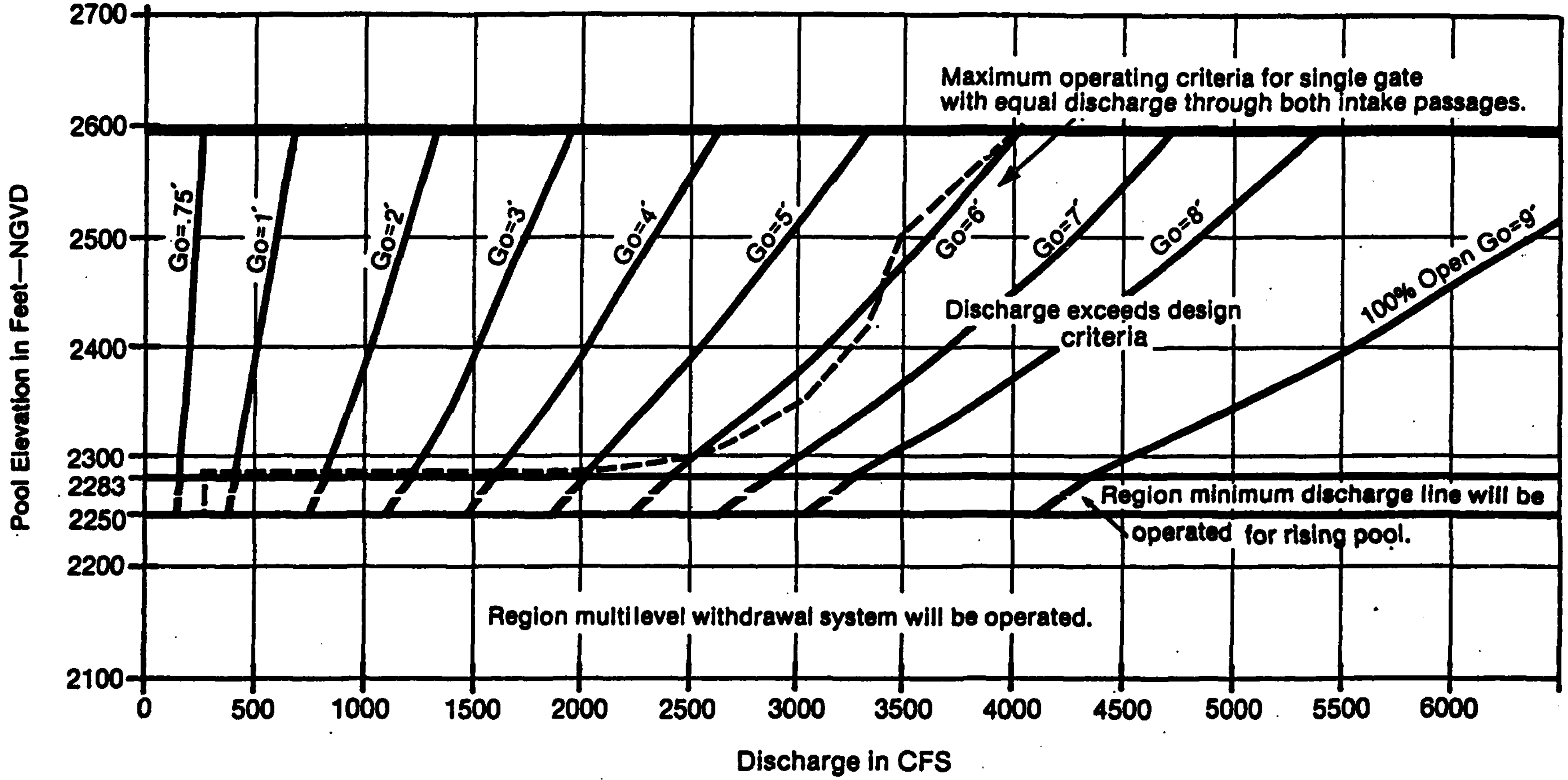


Figure 6-8 High level intake—rating curve for a single main operating gate.

SEVEN OAKS
Upstream Control
Main RO Gate Rating

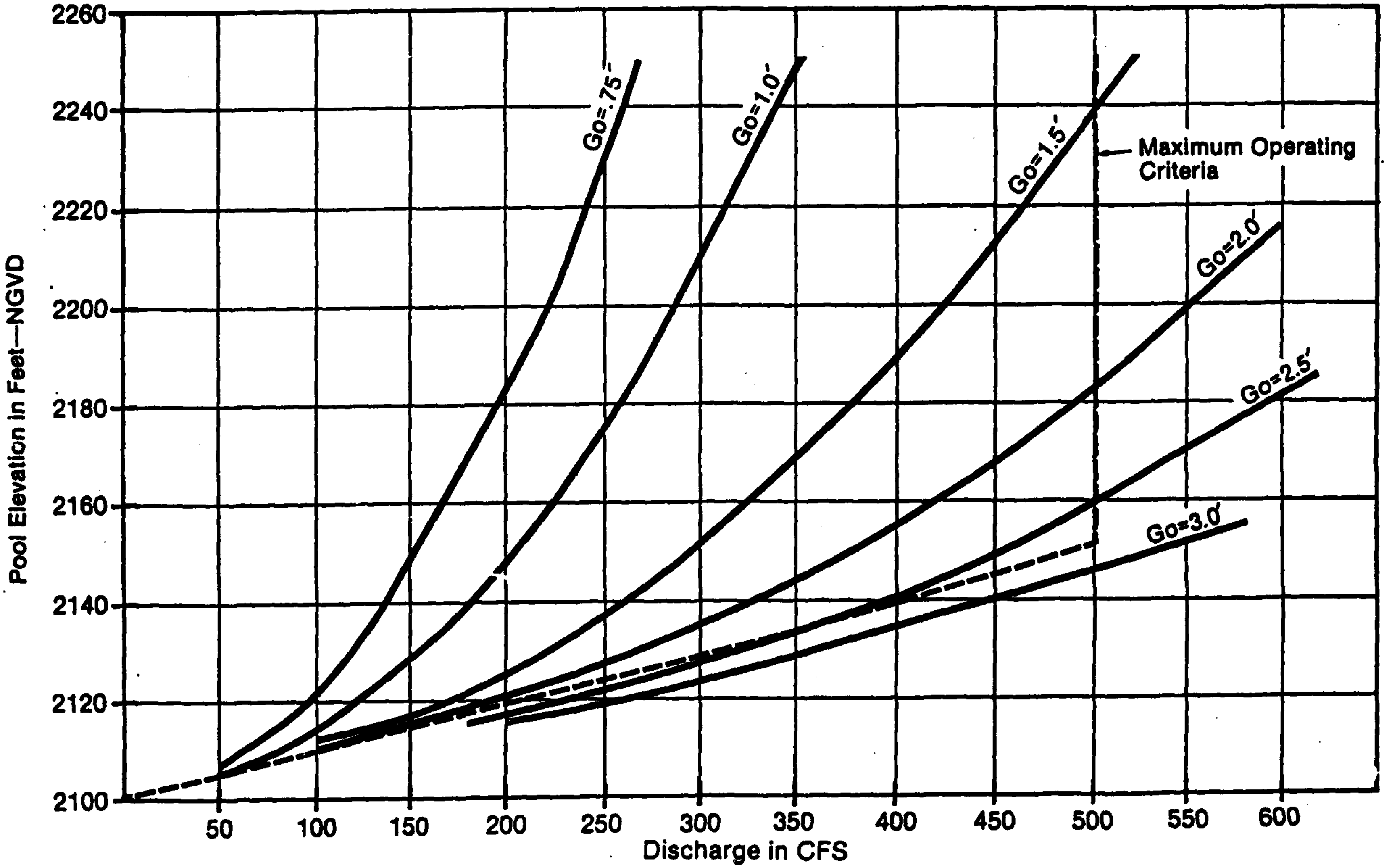


Figure 6-9 Multilevel withdrawal system—rating curve for a single main operating gate.

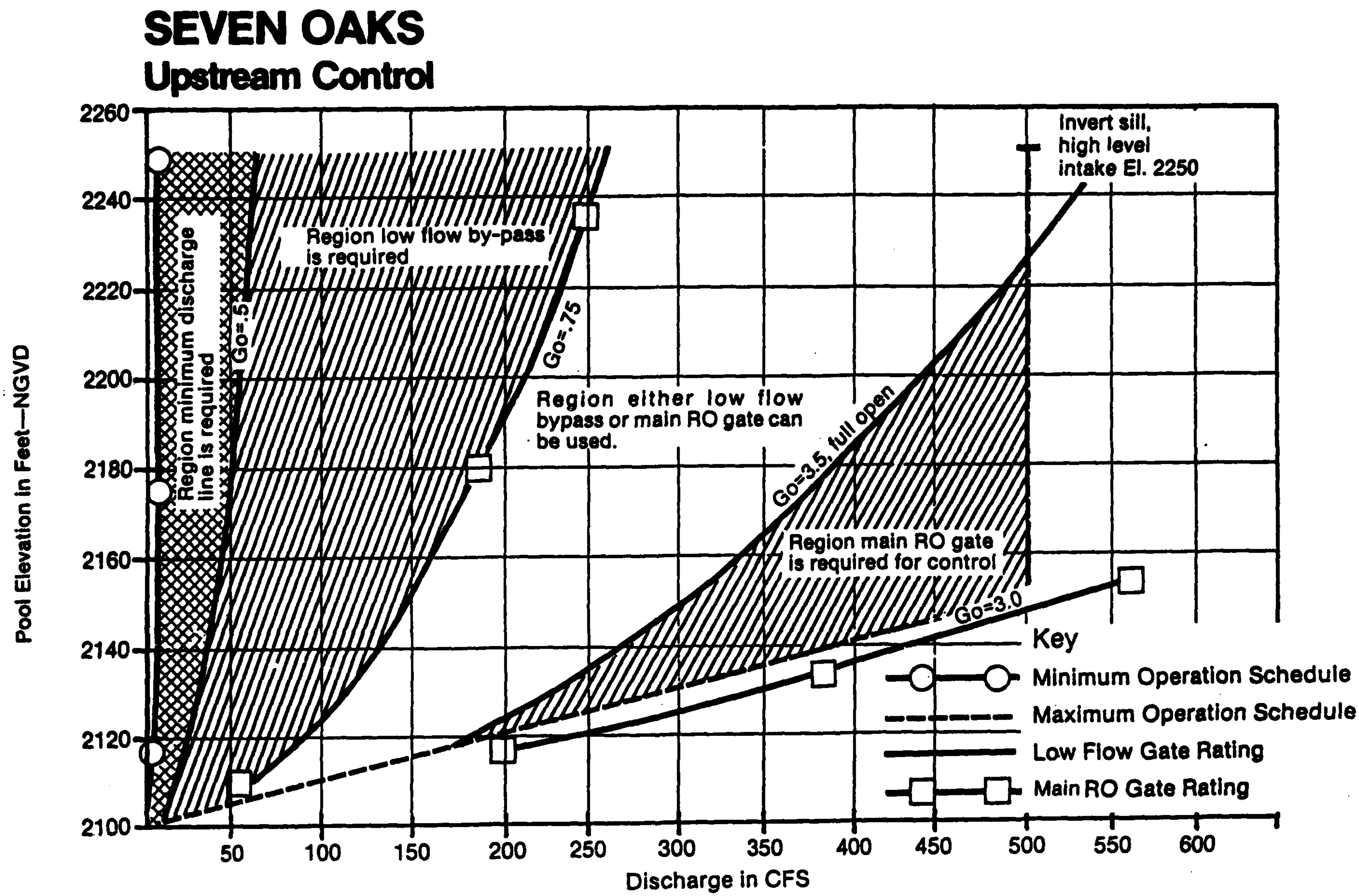


Figure 6-10 Multilevel withdrawal system—Regions minimum discharge line, low flow bypass, and main regulating outlets will be required.

Table 6-5. Upstream Control - Energy Loss Coefficients* (K Values) for High Level Intake and Main RO Gates

	Capacity	Rating	Velocity
Trash Struts	0.021 (1)	0.008 (2)	0.004 (3)
Drop Well	---- (4)	---- (5)	---- (6)
RO Intake Entrance	0.20 (7)	0.15	0.10
Friction, Gate Passageway	0.15 (8)	0.098 (9)	0.044 (10)
Gate Slot	0.01 (11)	0.01	0.01
Total	0.381	0.266	0.158

*Note that all loss coefficients are referenced to conduit proper upstream of gates.

(1) "Design of Small Dams," Bureau of Reclamation, page 366, eq. 11, trash struts 50 percent clogged.

(2) Same reference as for (1), trash struts 25 percent clogged.

(3) Same reference as for (1), trash struts 0 percent clogged.

(4) Darcy-Weisbach equation - equivalent f, given Manning's roughness coefficient n = 0.015. Found to be negligible, less than 0.0005.

(5) Same as for (4) except n = 0.012. Found to be negligible, less than 0.0005.

(6) Same as for (4) except n = 0.008. Found to be negligible, less than 0.0005.

(7) "EM 1110-2-1602," paragraph 3-7, page 3-5, and Lost Creek Dam computations.

(8) Darcy-Weisbach equation - equivalent f, given Manning's roughness coefficient n = 0.015.

(9) Same as for (8) except n = 0.012.

(10) Same as for (8) except n = 0.008.

(11) "EM 1110-2-1602," paragraph 3-7, page 3-5.

(3) Energy Loss Through Multilevel Withdrawal System. To develop the rating curves for the RO with flow entering the drop well through the multilevel withdrawal system, the head loss in feet through trashracks, ports, multilevel drop well, and wet well sluice gate passage was calculated for various discharges. The rating curve for the high level intake was then adjusted by adding the head loss to the pool elevation for the corresponding discharge. This adjustment was possible since losses through the high level trash struts and main drop well are negligible for the range of discharges passing through the multilevel withdrawal system. The trashrack loss coefficient value of 1.11 for the multilevel withdrawal system, referenced to velocity head through the net area of a trashrack for one row of ports, was determined for 25 percent clogging using equation 11, page 366, "Design of Small Dams," Bureau of Reclamation. An entrance loss coefficient K value of .98 was used for each port. This value was calculated by converting the average of discharge coefficients for a short tube (.82) and sharp edged orifice (.60) to an entrance loss coefficient. The entrance loss coefficient for a short tube and a sharp edged orifice were calculated to be .49 and 1.78, respectively. References used were "Handbook of Hydraulics," by Brater and King, page 4-19 through page 4-35, and "Design of Small Dams," Bureau of Reclamation, page 363. An exit loss coefficient K value of 1.0 was assumed for flow exiting ports into the multilevel drop wells. Friction loss coefficient, K value, in the multilevel drop well was estimated using an equivalent Manning's "n" value of .012. Entrance loss coefficient into the passage between drop wells, referenced to the velocity head in passageway, was estimated at .66. This value was based on coefficients of discharge for submerged tubes with square cornered entrances found in Kings Handbook, table 4-35, page 4-35, and "Design of Small Dams," page 363. Friction in the passage was found to be negligible because of the short passage length. For flow exiting the positive closure gate passage into the main drop well, an exitloss coefficient K value of 1.0 was assumed. Losses at the RO entrance and downstream to the vertical slide gates are the same as described in paragraph 6.5.c.2 and listed on table 6-5.

d. Low Flow.

(1) Low Flow Bypass. The low flow bypass conduit shown on plates 2-3 and 2-4 will eliminate the need to operate RO slide gates at openings of less than 9 inches. The entrance invert will be at El. 2,100. Elliptical curves are provided on intake passage roof and sides. A trashrack will be placed at the entrance of the low flow bypass to prevent material, which is small enough to pass through the trash struts for high level intake, from jamming the low flow slide gate. Trashrack openings are 16-inch square. Vertical and horizontal openings in trashracks are sized based on two-thirds of the gate width. Flow will be controlled with a 2-foot-wide by 3.5-foot-high vertical slide gate. An upstream emergency gate will be included. Provisions have been made for maintenance bulkheading. The low flow bypass will discharge flow into the main RO conduit about 80 feet downstream of the low flow bypass gate (see plate 2-4). The low flow bypass rating curves are shown on figure 6-11, high level intake, and figure 6-12, low pool multilevel withdrawal system. Figure 6-11 shows: minimum required flow conditions for Seven Oaks operation; discharge rating curves for a single 5-foot-wide by 9-foot-high main RO gate with 9-inch and 12-inch openings; and discharge rating curves for the RO low flow bypass conduit. The design condition for low flow bypass is discharges less than those which can be controlled with a main gate opening of 9 inches, but those discharges are required by the operation schedule. When the multilevel withdrawal system is being operated, the design discharge through the low flow bypass will be 260 cfs and occur at pool El. 2,250 (at pool El. 2,250, 260 cfs can be controlled with a main gate opening of 9 inches) as shown in figure 6-12. The bottom of the high level intake is at El. 2,250. When the high level intake is being used, the design discharge through the low flow bypass will be 490 cfs and occur at pool El. 2,598 (at pool El. 2,598, 490 cfs can be controlled with a main gate opening of 9 inches). The low flow bypass has been sized for more than minimum capacity, so there are regions where either the low flow gate or main gate can be used to control flow, as shown in figures 6-10 and 6-11. The energy loss coefficient K values used for design for operation of the high level intake are summarized in table 6-6. The average loss coefficient between the pool and the section just upstream of the gate, for the high level intake and relative to the area of low flow bypass intake conduit cross section,

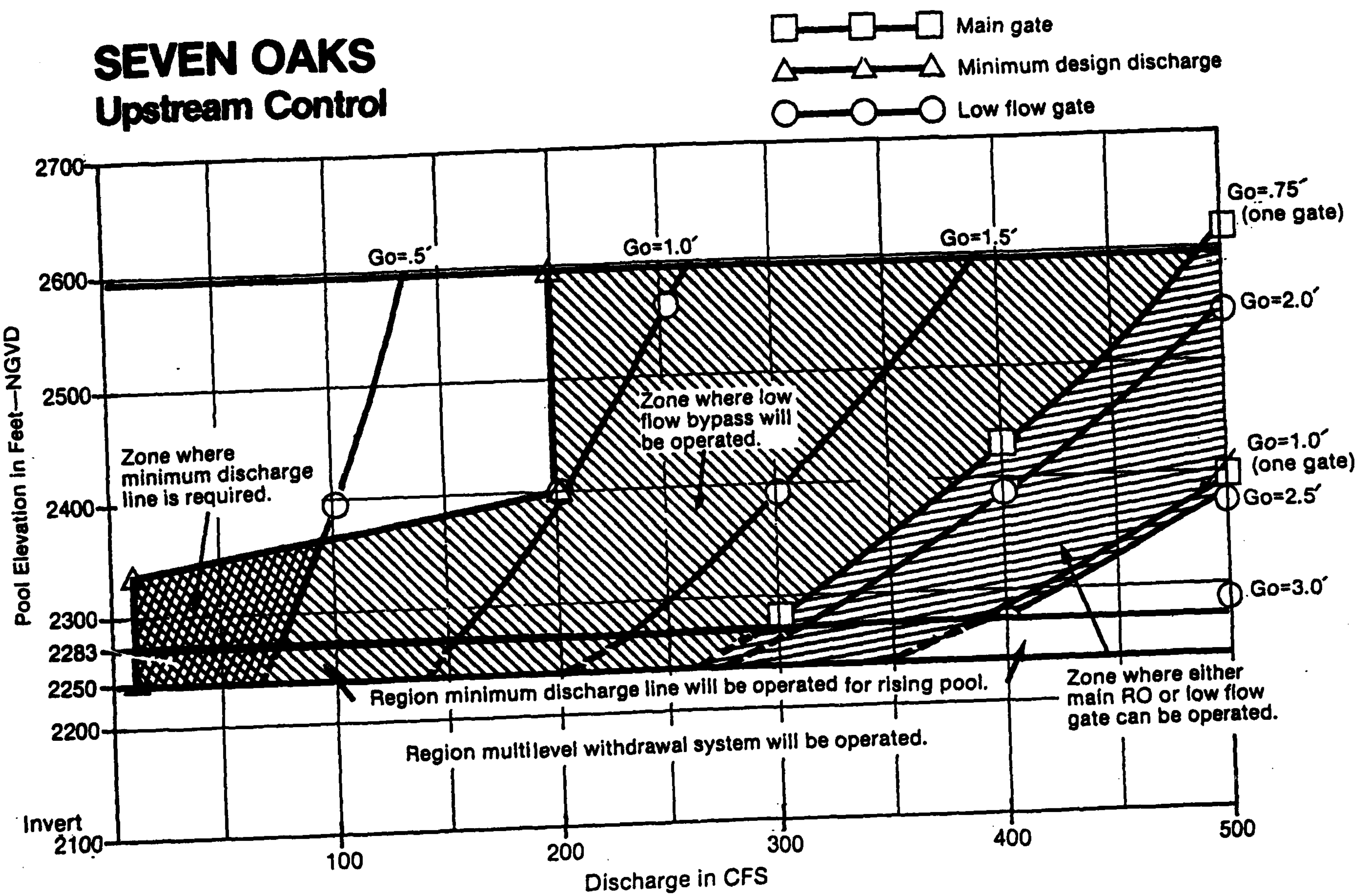


Figure 6-11 High level intake—rating curve for low flow bypass.

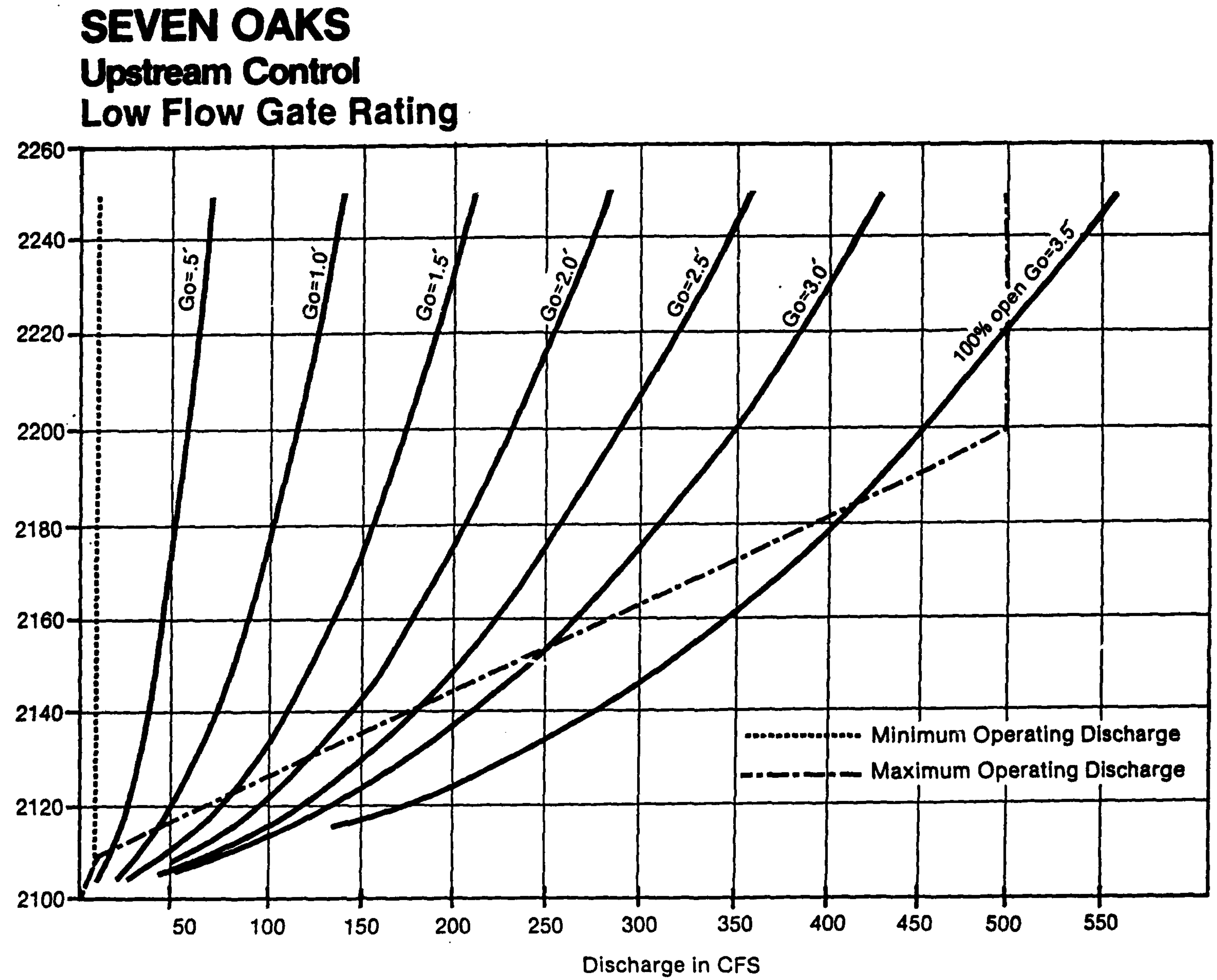


Figure 6-12 Multilevel withdrawal system—rating curve for low flow bypass

is .389. Head loss through the low pool multilevel withdrawal system has been accounted for as described in paragraph 6.5.c.3.

Table 6-6. Upstream Control - Energy Loss Coefficients* (K Values) for High Level Intake and Low Flow Bypass

	RATING
Trash Struts	---- (1)
Drop Well	---- (2)
Intake Entrance	0.15 (3)
Friction, Gate	
Passageway	0.23 (4)
Gate Slot	0.01 (5)
Total	0.39

*Note that all loss coefficients are referenced to conduit proper upstream of gates.

(1) "Design of Small Dams," Bureau of Reclamation, page 366, eq. 11, trash struts 25 percent clogged. Negligible.

(2) Darcy-Weisbach equation - equivalent f, given Manning's roughness coefficient n = 0.012. Negligible.

(3) "EM 1110-2-1602," paragraph 3-7, page. 3-5, and Lost Creek Dam computations.

(4) Darcy-Weisbach equation - equivalent f, given Manning's roughness coefficient n = 0.012.

(5) "EM 1110-2-1602," paragraph 3-7, page 3-5.

(2) Minimum Discharge Line. The minimum gate opening for the low flow bypass gate will be limited to 6 inches. There will be operating conditions when a minimum discharge line is required to control releases (see figures 6-10 and 6-11). This line will consist of a 16-inch-diameter pipe originating at the bottom of the multilevel withdrawal drop well. The line exits into the regulating conduit at the downstream end of the left transition

pier (see paragraph 6.5.g and plate 2-4). Flow is planned to be controlled with a disk gate valve. The minimum discharge line can be operated with the wet well sluice gate into the main drop well closed, allowing releases to be made while the main drop well is dewatered for inspection or repair. The rating curve for the minimum discharge line is shown on figure 6-13. An entrance loss coefficient of 0.20 was used for rating calculations. This assumes rounded edges at the intake entrance. An absolute roughness value of 0.00015 feet was based on figure 10-9, page 377, "Engineering Fluid Mechanics," Roberson/Crowe. A discharge line length of 120 feet was used for computations. Gate valve losses for various gate openings were determined using Hydraulic Design Criteria Chart 330-1. Cavitation may occur at partial gate openings and should be addressed in final design. Air supply to minimum discharge line and/or alternative gate valve types should be investigated at FDM level. An alternative design for the minimum discharge line is a 3-foot-diameter pressure pipe originating at the bottom of the multilevel withdrawal well that will carry flow the length of the RO works and will be regulated at the downstream end by a cone valve.

e. Aeration Scheme. Twelve-inch floor offsets and 6-inch wall offsets will be located 4.5 feet downstream of service gates for main intakes (see plate 2-4). A 6-inch floor offset and a 6-inch wall offset will be used for the low flow bypass. Offsets have been selected to ensure that air is insufflated into flow along boundaries. The aerated boundary will act as a cushion and provide protection to keep vapor cavities from collapsing against concrete surfaces. Offsets were selected over air slots because: offsets will not fill with water as air slots can; offsets will not fill with sediment; offsets provide more water surface for aeration; offsets separate flow surfaces from jet for longer distances, entraining more air; offsets are less critical to construct; and offsets increase the height and width of the conduit which is favorable in transitioning from the gate passages to the main tunnel. Preliminary design of offsets was made using recommendations in "Hydraulic Model Studies of Chute Offsets, Air Slots, and Deflectors for High-Velocity Jets," REC-ERC-73-5, G. L. Beichley, Bureau of Reclamation, March 1973. The cavitation potential was analyzed at the Bureau of Reclamation Engineering and Research Center in Denver, Colorado. The point at which the jet first strikes

SEVEN OAKS
Upstream Control
Minimum Discharge Line Rating

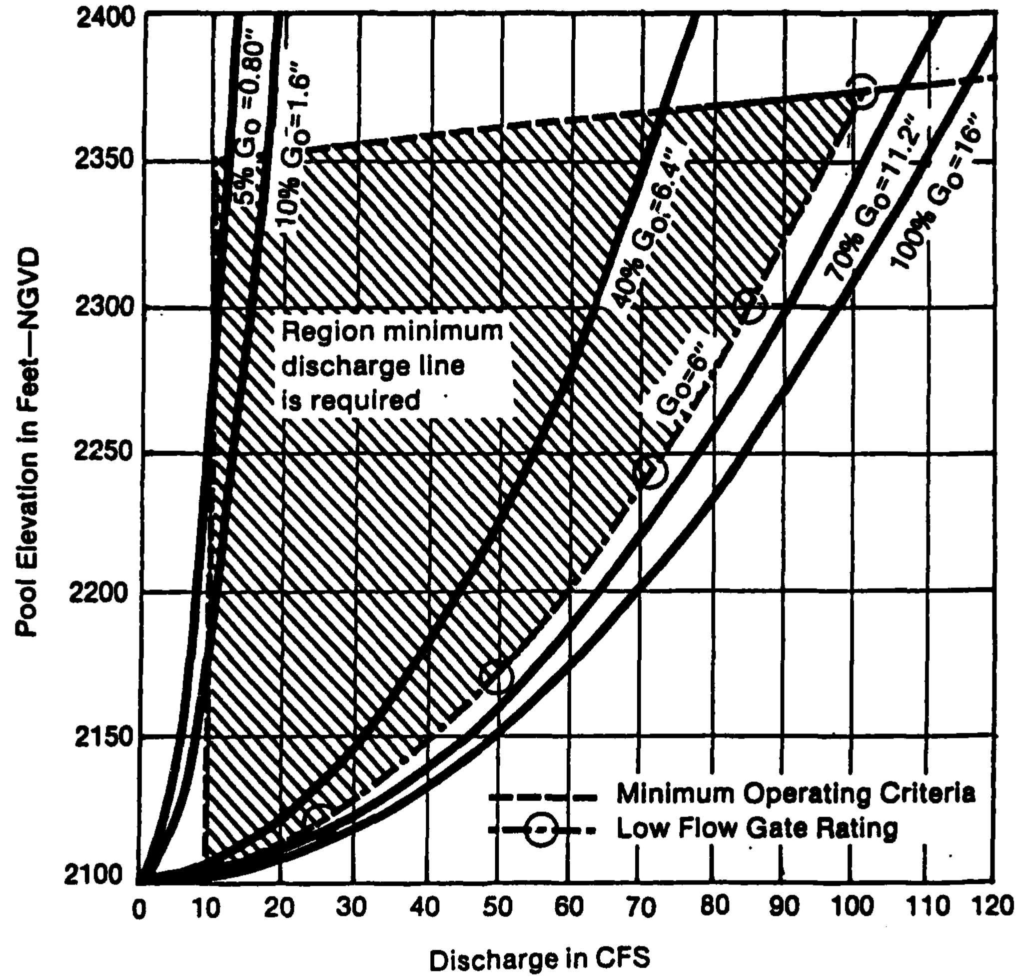


Figure 6-13 Minimum discharge line rating curves.

the invert was computed to be 34 feet downstream from offsets for a discharge of 8,000 cfs. It appears that the offsets will provide sufficient protection against cavitation. Provision for a secondary slot approximately 500 feet downstream of offset has been provided, however. Slots are economically preferable at this downstream location because no change in area is needed. Model studies will help determine location and need for secondary air slots. Note that air supply to minimum discharge line will need to be investigated at FDM level.

f. Air Demand.

(1) General. Air requirements were estimated using the four different methods listed below:

(a) EM 1110-2-1602 design guidance.

(b) Bureau of Reclamation, Engineering Research Center, computer program on aeration.

(c) Assumed velocity distribution for air in RO conduit.

(d) Libby Dam model and prototype test results.

Results from the four methods were compared and used to estimate the total air demand, where total air demand is a combination of: air entrained in water; and air moving above water surface (surface air demand) due to shear field created by relative movement of water and air.

(2) Main Regulating Outlet Conduits. The maximum air demand is expected to fall in the range of 3,200 to 5,200 cfs per main intake conduit. The upper limit was used for design. This gives a maximum ratio, of air demand to water discharge, of 1.3. Two air passages, each with an area of 50 square feet, start at the downstream end of the RO tunnel and end in the vicinity of the hydraulic slide gates. A 6-foot by 6-foot air vent will be located on the roof of each intake conduit directly above wall offsets, limiting the maximum air velocity to 150 fps (see plate 2-4). The downstream intake for air supply has been designed to ensure that velocities are less

than 30 fps. Maximum headloss through the air passage system was calculated to range between 1.5 and 2.4 feet of water which is higher than recommended in the EM. The maximum head loss through air passage system was calculated using conservative loss coefficients and high air demand. If care is used in design of bends, transitions, etc., at FDM level, it should be possible to reduce head loss. In this case, due to large costs associated with the size of air vents, an exception was made to the general EM guidance.

(3) Low Flow Bypass. Four hundred ninety cfs of water is the largest discharge which will have to be passed through the low flow bypass (see figure 6-11). Based on results from the four methods listed above, the maximum air demand might be as high as 1.3 times the maximum water discharge. The validity of this factor for the low flow bypass should be checked at the feature design level, since the factor of 1.3 is based on flow through the main RO conduit. An air demand of 640 cfs was used for design. Since water discharges of 490 cfs are expected at much higher frequencies than the design discharge (8,000 cfs), air velocities have been limited to less then 150 fps for the low flow system, the maximum air velocity recommended in EM-1110-1602. Justification for this reduction in maximum air velocity should be investigated in more detail at the FDM level. Two 2-foot-diameter conduits will supply air from the main tunnel. Air will be distributed from the roof at the aeration offsets using a 2.33-foot by 3-foot plenum. The same downstream intake for air supply will be shared by both low flow bypass and main intake conduits.

g. Transition. The two RO conduits, each 5 feet wide by 9 feet high, and the 2-foot-wide by 3.5-foot-high low flow bypass conduit symmetrically transition to an 18-foot oblong section (see plate 2-4). The transition takes place over a length of 230 feet beginning at the offset section. Criteria used in the design of the transition are jet trajectory equations, EM 1110-2-1602 guidance, information provided by the Bureau of Reclamation, and investigations and computations from existing projects. Five feet downstream of the RO gates, the 5-foot by 9-foot section expands to a 6-foot-wide by 17-foot-high section via roof expansion, a 1-foot offset on the floor, and a 0.5-foot offset on each wall. The conduit floor and walls will be steel lined from the RO gates downstream to the offsets. Two piers separate the conduits

for a distance of 60 feet downstream of the offset. To ensure that offsets provide sufficient separation of water surface from conduit walls and floors, the tunnel width remains constant at 24 feet for a distance of 80 feet downstream of the offsets. The RO conduits expand to 8 feet wide, the low flow bypass conduit expands to 5 feet wide, and the pier thickness decreases to 1.5 feet. A 20-foot tangent section is provided downstream of the pier nose, then the conduit transitions from a rectangular section to an oblong section over the next 150 feet. Using guidance in EM 1110-2-1602, the minimum transition length, from rectangular to circular section, was calculated to be 100 feet. The length was increased 50 percent based on prototype experience. A hydraulic model study should be conducted at the FDM level to confirm that the entire transition will be hydraulically acceptable. An energy loss (minor loss) through transition of 0 percent was used to calculate information required for design of energy dissipator (maximum velocity), and an energy loss of 20 percent was used for capacity design (maximum depth). For friction, Manning's "n" values of 0.008 and 0.015 were used for velocity and capacity design, respectively. These computations, from the RO gates downstream to the end of the transition, are summarized in table 6-7.

Table 6-7. Upstream Control - 18-Foot-Oblong Tunnel - Velocity and Capacity Design Computations Downstream from Regulating Outlet Gates to End of Transition

Variable	Maximum Velocity	Maximum Depth
Maximum discharge (total)	8,000 cfs	8,000 cfs
Pool elevation	2598 ft	2580 ft
Gate opening	5.9 ft	6.2 ft
Depth at vena contracta	4.6 ft	4.8 ft
Velocity at vena contracta	174.7 ft/s	165 ft/s
Specific energy at vena contr.	478 ft	433 ft
N-value through transition	0.008	0.015
Percent energy loss due to trans.	0 %	20 %
Percent energy loss from friction	11 %	31 %
Specific energy at end of trans.	428 ft	212 ft
Depth at end of transition	4.4 ft	5.7 ft
Velocity at end of transition	165.1 ft/s	115.3 ft/s

h. Regulating Outlet Conduit Design. The tunnel has an oblong cross section consisting of two 18-foot-diameter semi-circles separated by an 18-foot-wide by 14-foot-high rectangular section. The lower 17 feet of the 32-foot-high tunnel are available for flow. The upper 15 feet will be used for downstream access and air passage. The conduit was sized using capacity energy loss coefficients and assuming 50 percent bulking due to air entrainment. The conduit is designed to pass 8,000 cfs with open channel flow. Two flow conditions were examined: maximum velocity for design of energy dissipator; and maximum depth for design of system capacity. Depths at the upstream end of the tunnel were determined on the basis of maximum and minimum energy losses through the transition. These depths were then used in the CORPS H6209 program to calculate water surface profiles through the conduits. Manning's "n" values of 0.008 and 0.015 were used for velocity and capacity design, respectively. Calculations were made for a tunnel length of 1,525 feet. For capacity design the maximum depth in the tunnel is 9.0 feet, for a discharge of 8,000 cfs without air entrainment. Up to a 50 percent increase in volume due to air entrainment is expected. This may be conservative since air has been assumed to stay entrained in flow along the entire conduit length, even though some air may escape from flow as water velocities decrease downstream. This percentage should be used for design work until further study is performed, however. Bulking will increase maximum depth of flow (air-water mixture) to 12.5 feet (74 percent of available conduit height). An advantage of the oblong cross-section geometry is that conduit height can easily be decreased by changing height of rectangular center, if assumed bulking is later found to be overconservative. Minimum energy loss coefficients were used to calculate the maximum velocity at the portal exit of 125 fps. Numerical results of analysis for velocity and capacity design are listed in table 6-8, and a sketch of water surface profiles is shown on figure 6-14. The energy grade line and pressure grade line for the velocity and capacity analyses are shown on figures 6-15 and 6-16.

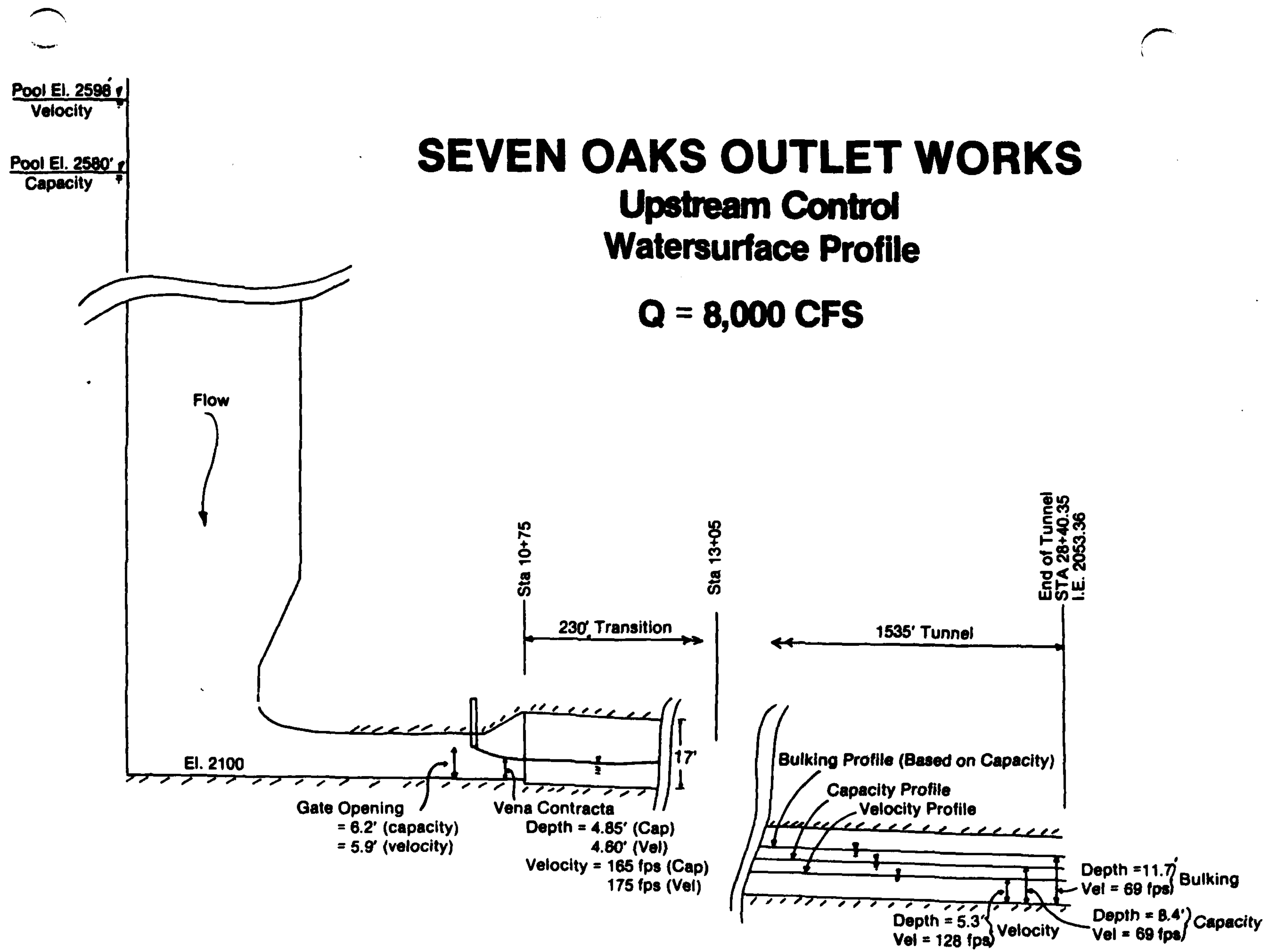


Figure 6-14 Flow profile for capacity, velocity, and bulking—18 Ft. oblong.

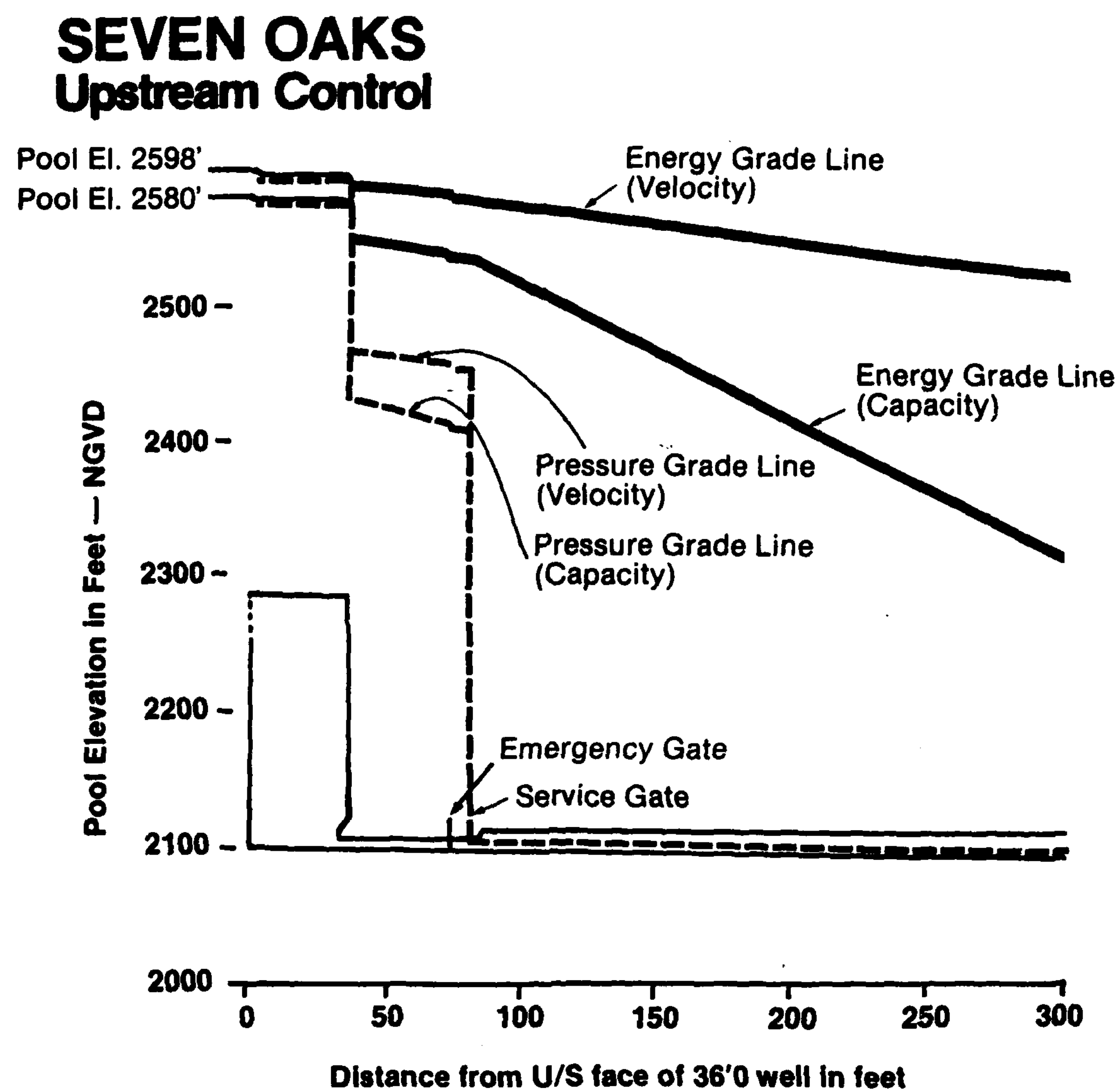


Figure 6-15 EGL and PGL for Intake Area, Q=8000 cfs

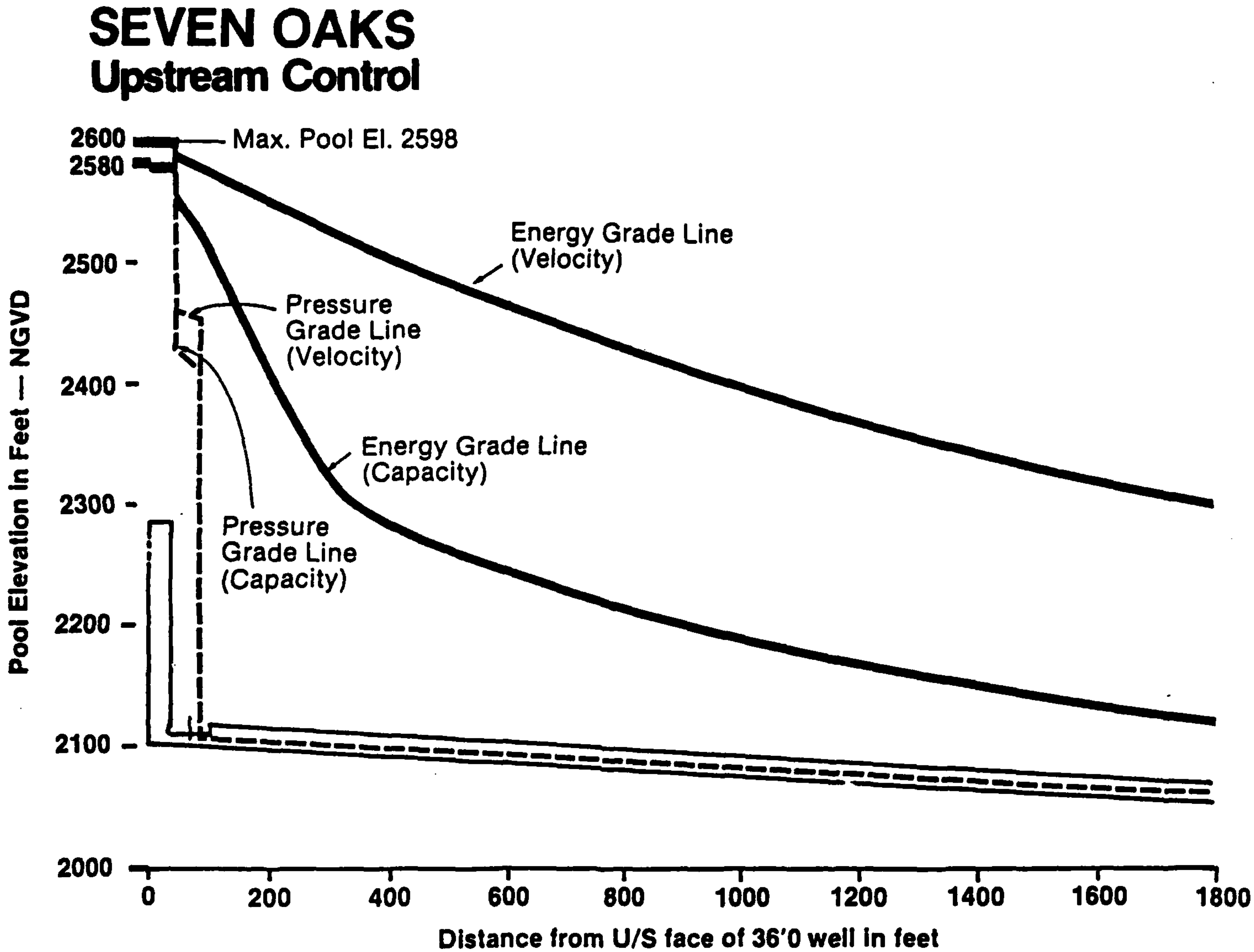


Figure 6-16 EGL and PGL for Regulating Outlet Works, Q=8000 cfs

Table 6-8. Upstream Control - 18-Foot-Oblong Tunnel Alternative - Velocity and Capacity Design Computations Downstream from End of Transition to Portal Exit

Variable	Maximum Velocity	Maximum Depth
Maximum discharge (both conduits operating)	8,000 cfs	8,000 cfs
Pool elevation	2,598 ft	2,580 ft
Depth at end of transition	4.4 ft	5.7 ft
Velocity at end of transition	165.1 ft/s	115.3 ft/s
N-value through conduit	0.008	0.015
Velocity at end of conduit	124.5 ft/s	62.9 ft/s
Depth at end of conduit	5.4 ft	9 ft
Depth at end of conduit (with 50 percent bulking)	7.3 ft	12.5 ft
Percent of conduit height (with 50 percent bulking)	42.8 %	73.7 %

6.6 Downstream Control.

a. General. The downstream control alternative was analyzed for a steel conduit within a tunnel. This alternative was investigated for reasons discussed in paragraph 1.6. Advantages and disadvantages are listed in section 5. Details of the anlysis provided are described in the following text.

b. Regulating Outlet Intake. The RO entrance invert will be at El. 2,100 feet. One RO intake passage, 11-foot square, will be used. Provisions have been made for maintenance bulkheading. Geometry of entrance curves is based on the design of Lost Creek Dam. The RO works at Lost Creek were model studied, as outlined in Technical Report No. 140-1, "Outlet Works for Lost Creek Dam, Rogue River, Oregon," Division Hydraulic Laboratory, U.S. Army Corps of Engineers, North Pacific Division. The Lost Creek project has operated successfully since 1977. Lost Creek has similar vertical entrance

conditions and the model study indicated there would be no adverse pressures on the system. A compound ellipse has been selected for the roof curve, approximately a two on three ellipse for the upstream portion, and approximately a one on three ellipse for the downstream portion. This curve is designated type 5 for entrances flared in three directions in Technical Memorandum No. 2-428, Report No. 2, "Investigation of Entrances Flared in Three Directions and in One Direction," Waterways Experiment Station. The intake curve has been extended 9 feet into the wet well to provide an acceptable transition from the circular well to a flat front entrance. The side curves are conventional one on three ellipses. Horizontal and vertical entrance curves are based on width and height, respectively, of the RO intake conduit. The 11-foot-square intake passage will transition to an 11-foot-diameter conduit, over a distance of 20 feet. The transition design is based on guidance provided in EM 1110-2-1602, paragraph 4.20.c.

c. Pressure Conduit. An 11-foot-diameter pressure conduit will be used. At the design discharge of 8,000 cfs, the velocity in the steel conduit will be 84.1 fps. The smooth pipe curve from the Moody diagram was used to calculate friction loss in the pressure conduit for maximum velocities. Absolute roughness values for rating and capacity computations are .00015 feet and .003 feet, respectively, and were based on steel in new condition and heavily rusted. Friction values obtained from the Moody diagram are .0063, .0086, and .015 for velocity, rating, and capacity computations, respectively. The frictional loss coefficients to be used in the Darcy-Weisbach equation were calculated to be .964, 1.32, and 2.3 relative to the 11-foot-diameter conduit and a pipe length of 1,683 feet. To evaluate the impact on project operation if an earthquake occurs, the maximum discharge was estimated for a break in the 11-foot conduit just upstream of the downstream transition (see paragraph 6.6.d). Discharge would be less for breaks further upstream, due to flow restriction from outer 18-foot horseshoe tunnel. The maximum estimated discharge is 11,800 cfs, using minimum loss coefficients for analysis.

d. Downstream Transition. The 11-foot-diameter main RO conduit transitions into two 5-foot-wide by 9-foot-high gate passages over a length of 55 feet (see plate 3-6). Criteria used in the transition design are EM 1110-2-1602 guidance and investigations of existing projects. The 11-foot-diameter conduit transitions to an 11-foot-square section over 20 feet. The square section symmetrically transitions to an 18-foot-wide by 9-foot-high section with an 8-foot-wide splitter pier in the center, leaving two 5-foot-wide by 9-foot-high gate passages. The pier nose begins 15 feet downstream of the 11-foot-square section, and has an end radius of 1.0 foot. The pier width expands to 8 feet over a length of 20 feet. The service gates are 25 feet downstream of the end of the transition. A loss coefficient (minor loss) through the transition of .10, .2, and .4, relative to the area of the gate passages, was used for velocity, rating, and capacity design, respectively. A hydraulic model study should be conducted at the FDM level to confirm that the entire transition will be hydraulically acceptable.

e. Regulating Outlet Gate Rating.

(1) General. The RO rating curves for various gate openings are shown on figure 6-17 (high level intake) and figure 6-18 (multilevel withdrawal system). Figure 6-19 shows regions where minimum discharge line, low flow bypass, and main regulating outlet are required when multilevel withdrawal system is used. Average values of loss coefficients have been used to approximate actual operating conditions. Minimum, average, and maximum loss coefficients have been calculated from the pool to the regulating gates and were used for velocity, rating curve, and capacity design, respectively.

(2) Energy Loss Coefficients for High Level Intake. Average loss coefficients for the high level intake have been computed and are summarized in table 6-9. Trash strut loss coefficients have been determined for velocity, capacity, and rating, and are described in paragraph 6.2.b. Wet well friction loss coefficients were calculated, but were found to be negligible as stated in paragraph 6.3.b. RO intake losses are as shown in table 6-9. The smooth pipe curve and absolute roughness values of .00015

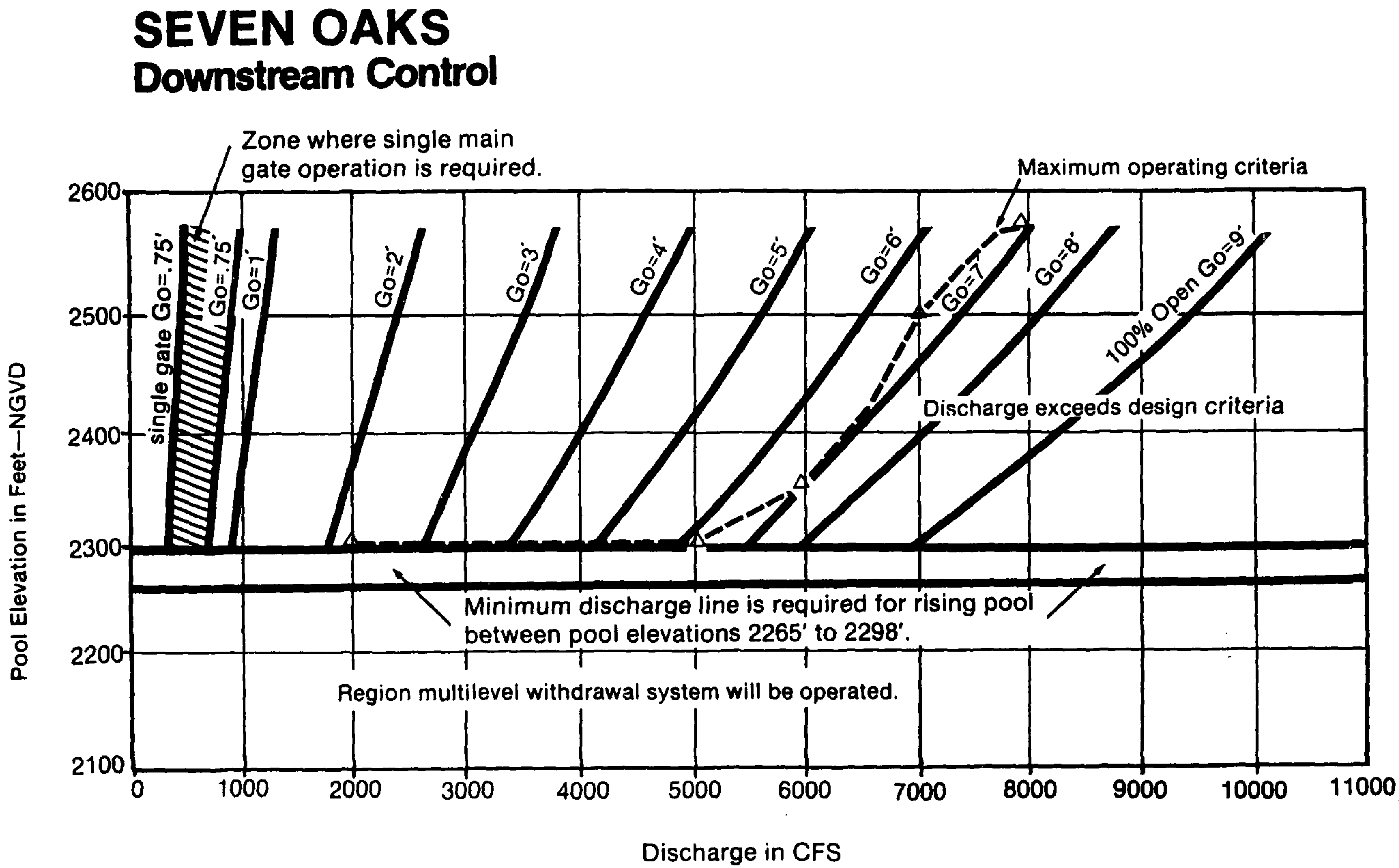


Figure 6-17 High level intake-Main gate rating, 2 gates operating with balanced operation.

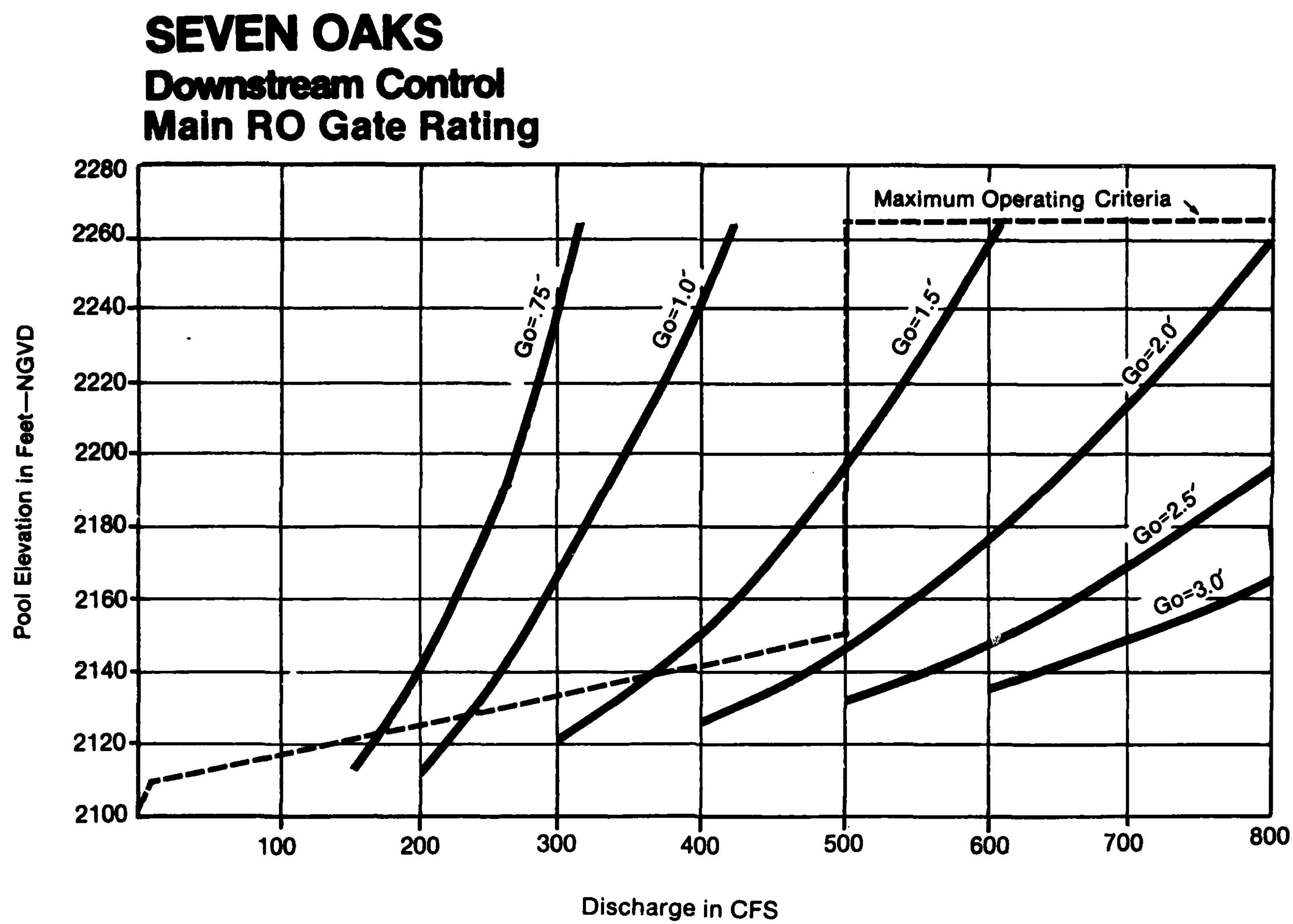


Figure 6-18 Multilevel withdrawal system—rating curve for a single main operating gate.

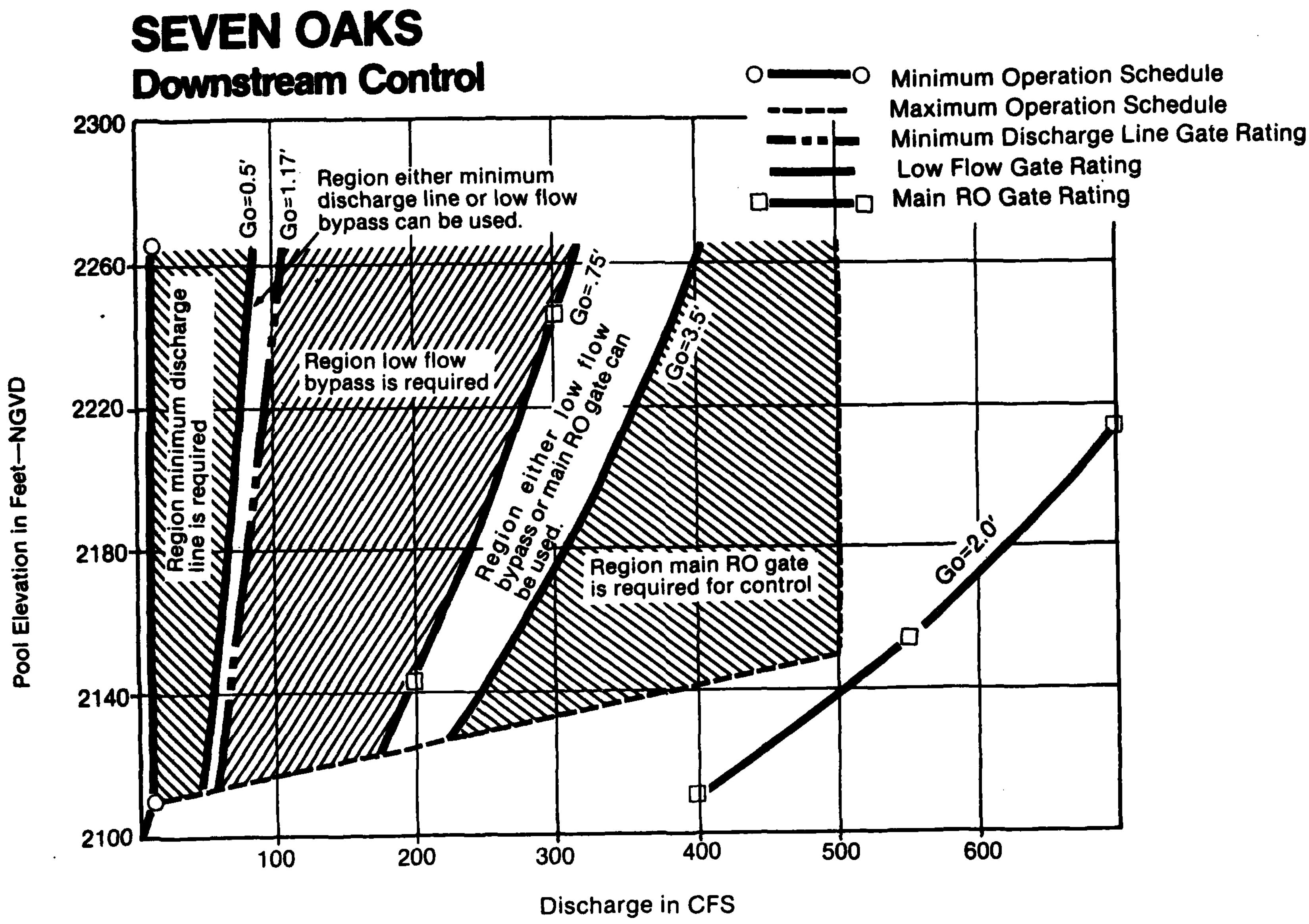


Figure 6-19 Multilevel withdrawal system—Regions minimum discharge line, low flow bypass, and main regulating outlets will be required.

Table 6-9. Downstream Control - Energy Loss Coefficients* (K Values) for High Level Intake and Main RO Gates

	Capacity	Rating	Velocity
Trash Struts	0.025 (1)	0.010 (2)	0.005 (3)
Wet Well	---- (4)	---- (5)	---- (6)
RO Intake Entrance	0.111 (7)	0.083	0.055
Bulkhead Slot	0.006 (8)	0.006	0.006
Upstream Transition	0.034 (9)	0.009	----
Friction, Pressure Conduit	2.064 (10)	1.185 (11)	0.865 (12)
Downstream Transition	0.40 (13)	0.20	0.10
Gate Slot	0.01 (14)	0.01	0.01
Total	2.650	1.503	1.041

*Note that all loss coefficients are referenced to the gate passages upstream of gates, with both gates operating equally.

(1) "Design of Small Dams," Bureau of Reclamation, page 366, eq. 11, trash struts 50 percent clogged.

(2) Same reference as for (1), trash struts 25 percent clogged.

(3) Same reference as for (1), trash struts 0 percent clogged.

(4) Darcy-Weisbach equation - equivalent f, given Manning's roughness coefficient n = 0.015. Found to be negligible, less than 0.0005.

(5) Same as for (4) except n = 0.012. Found to be negligible, less than 0.0005.

(6) Same as for (4) except n = 0.008. Found to be negligible, less than 0.0005.

(7) "EM 1110-2-1602," paragraph 3-7, page 3-5, and Lost Creek Dam computations.

(8) "EM 1110-2-1602," paragraph 3-7, page 3-5.

(9) "EM 1110-2-1602," Plate C-9 and "Engineering Fluid Mechanics," Roberson/Crowe, page 384.

(10) "Handbook of Hydraulics," Brater and King, pages 6-12, and "Engineering Fluid Mechanics," page 376, f = 0.0150, D = 11 feet, L = 1,683 feet.

(11) Same as (9) except f = 0.0086.

(12) Same as (9) except f = 0.0063.

(13) "Hydraulic Design Chart" 221-1/3.

(14) "EM 1110-2-1602," paragraph 3-7, page 3-5.

and .003 feet were used to compute friction "f" values for the pressure conduit as described in paragraph 6.6.c. Minor loss coefficients due to the transition from the pressure conduit to two gate passages are described in paragraph 6.6.d. The total loss coefficients for velocity, rating, and capacity are shown in table 6-9. All loss coefficients are relative to the velocity through the gate passages.

(3) Energy Loss Through Multilevel Withdrawal System. To develop the rating curves for the RO with flow entering the wet well through the multilevel withdrawal system, the head loss in feet through trashracks, ports, multilevel drop well, and wet well sluice gate passage was calculated for various discharges. The rating curve for the high level intake was then adjusted by adding the head loss to the pool elevation for the corresponding discharge. This adjustment was possible since losses through the high level trash struts and main wet well are negligible for the range of discharges passing through the multilevel withdrawal system. The trashrack loss coefficient value of 1.11 for the multilevel withdrawal system, referenced to velocity head through the net area of a trashrack for one row of ports, was determined for 25 percent clogging using equation 11, page 366, "Design of Small Dams," Bureau of Reclamation. An entrance loss coefficient K value of .98 was used for each port. This value was calculated by converting the average (.71) of discharge coefficients for a short tube (.82) and sharp edged orifice (.60) to an entrance loss coefficient. References used were "Handbook of Hydraulics," by Brater and King, page 4-19 through page 4-35, and "Design of Small Dams," Bureau of Reclamation, page 363. An exit loss coefficient K value of 1.0 was assumed for flow exiting ports into the multilevel wet wells. Friction loss coefficient, K value, in the multilevel wet well was estimated using an equivalent Manning's "n" value of .012. Entrance loss coefficient into the passage between wet wells, referenced to the velocity head in passageway, was estimated at .66. This value was based on coefficients of discharge for submerged tubes with square cornered entrances found in Kings Handbook, table 4-35, page 4-35, and "Design of Small Dams," page 363. Friction in the passage was found to be negligible because of the short passage length. For flow exiting the wet well sluice gate passage into the main wet well, an exit loss coefficient K value of 1.0 was assumed. Losses at the RO entrance and downstream to the vertical slide gates are the same as described in paragraph 6.6.e(2) and listed on table 6-9.

f. Low Flow.

(1) Low Flow Bypass. The low flow bypass conduit shown on plates 3-4 and 3-6 will eliminate the need to operate RO slide gates at openings of less than 9 inches. The entrance invert will be at El. 2,100. Elliptical curves are provided on intake passage roof and sides. A trashrack will be placed at the entrance of the low flow bypass to prevent material, which is small enough to pass through the trash struts for high level intake, from jamming the low flow slide gate. Trashrack openings are 16-inch square. Vertical and horizontal openings in trashracks are sized based on two-thirds of the gate width. Provisions have been made for upstream maintenance bulkheading. The 3.5-foot-square intake passage will transition to a 3.5-foot-diameter pipe. The 3.5-foot-diameter steel pipe will carry flow under pressure a distance of about 1,680 feet to the low flow gate. The circular pipe transitions to a 2-foot-wide by 3.5-foot-high rectangular section. Flow will be controlled with a 2-foot-wide by 3.5-foot-high vertical slide gate. The low flow bypass rating curves are shown on figure 6-20, high level intake, and figure 6-21, low pool multilevel withdrawal system. Figure 6-20 shows: minimum required flow conditions for Seven Oaks operation; discharge rating curves for a single 5-foot-wide by 9-foot-high main RO gate with a 9-inch opening; and discharge rating curves for the RO low flow bypass conduit. The design condition for low flow bypass is discharges less than those which can be controlled with a main gate opening of 9 inches, but those discharges which are required by the operation schedule. When the multilevel withdrawal system is being operated, the design discharge through the low flow bypass will be 310 cfs and occur at pool El. 2,265 (at pool El. 2,265, 310 cfs can be controlled with a main gate opening of 9 inches) as shown in figure 6-19. The bottom of the high level intake is at El. 2,265. When the high level intake is being used, the design discharge through the low flow bypass will be 500 cfs and occur at pool El. 2,580 (at pool El. 2,580, 500 cfs can be controlled with a main gate opening of 9 inches) as shown in figure 6-20. The low flow bypass has been sized for more than minimum capacity of the main gates, so there are regions where either the low flow gate or main gate can be used to control flow, as shown in figures 6-19 and 6-20. The energy loss coefficient K value, used for design for operation of the high level intake are summarized in table 6-10. The average loss coefficient between the pool and the section just upstream of the

SEVEN OAKS
Downstream Control

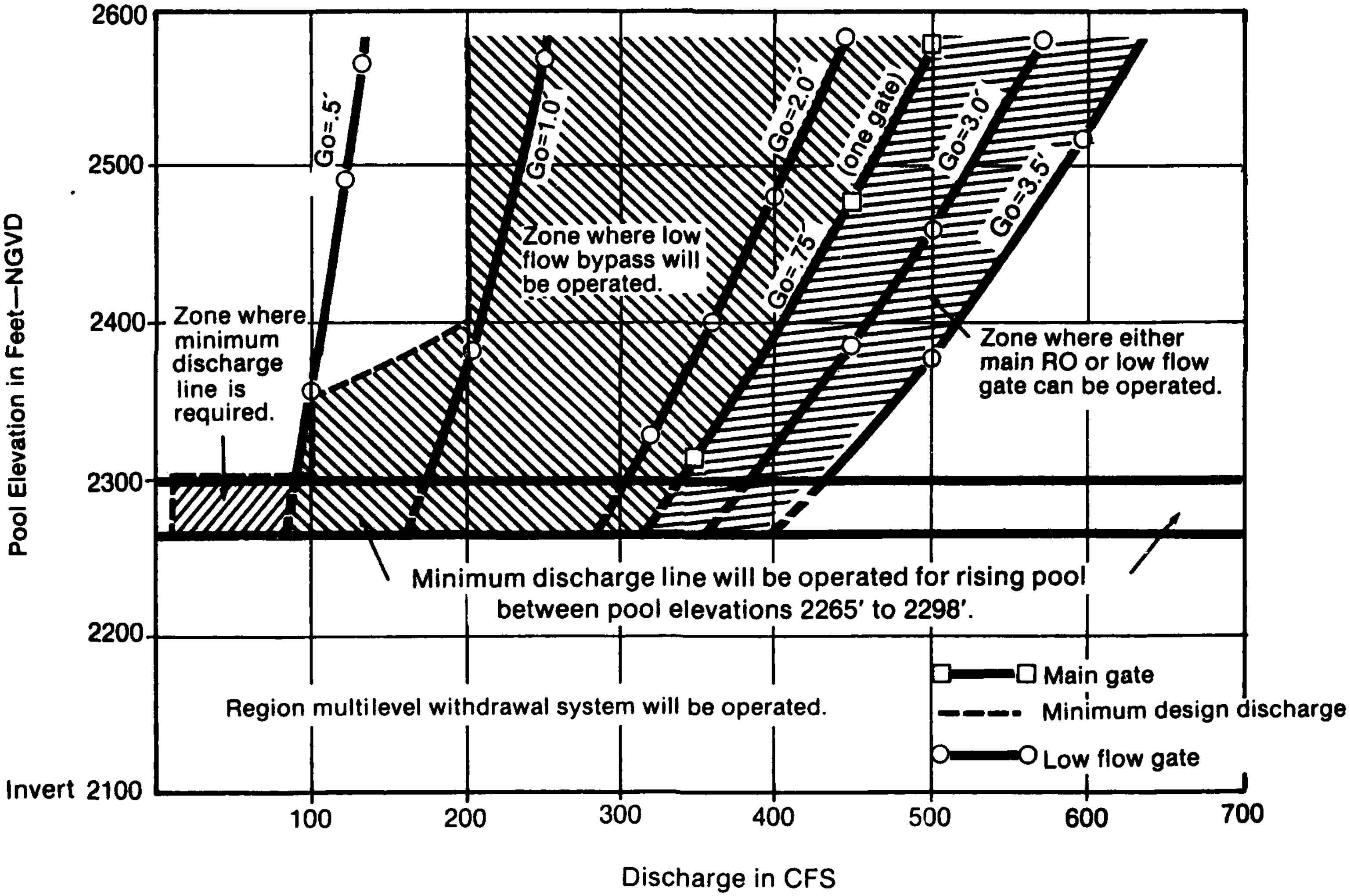


Figure 6-20 High level intake—rating curve for low flow bypass.

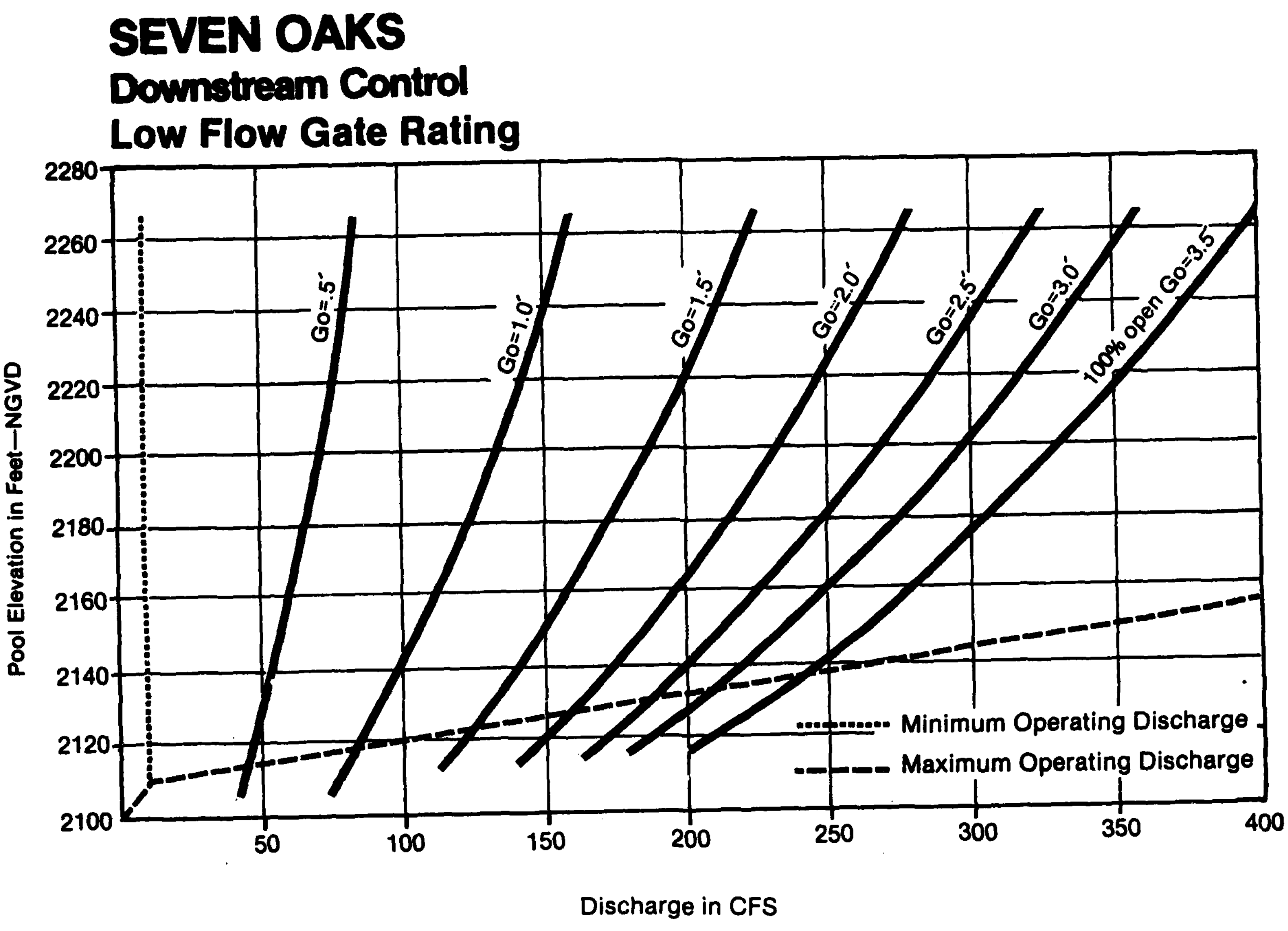


Figure 6-21 Multilevel withdrawal system—rating curve for low flow bypass

Table 6-10. Downstream Control - Energy Loss Coefficients* (K Values) for High Level Intake and Low Flow Bypass

	RATING	
Trash Struts	----	(1)
Wet Well	----	(2)
Intake Entrance	0.033	(3)
U/S Transition	----	(4)
U/S Bends	0.159	(5)
Tunnel Friction	2.70	(6)
D/S Bends	0.106	(7)
Gate Slot	0.01	(8)
Total	3.01	

*Note that all loss coefficients are referenced to conduit proper upstream of gates.

(1) "Design of Small Dams," Bureau of Reclamation, page 366, eq. 11, trash struts 25 percent clogged. Negligible.

(2) Darcy-Weisbach equation - equivalent f, given Manning's roughness coefficient n = 0.012. Negligible.

(3) "EM 1110-2-1602," paragraph 3-7, page 3-5, and Lost Creek Dam computations.

(4) "EM 1110-2-1602," plate C-9. Negligible.

(5) "Handbook of Hydraulics," Brater and King, page 6-24.

(6) "Handbook of Hydraulics," page 6-12 and "Engineering Fluid Mechanics," Roberson/Crowe, page 376, absolute roughness = 0.00015 feet, f = .0106, D = 3.5 feet, L = 1,680 feet.

(7) Same as (5).

(8) EM 1110-2-1602," paragraph 3-7, page 3-5.

gate, for the high level intake and relative to the area of low flow bypass intake conduit cross section, is 3.01. Head loss through the low pool multilevel withdrawal system has been accounted for as described in paragraph 6.6.e(3).

(2) Minimum Discharge Line. The minimum discharge line shown on plates 3-4 and 3-6 will eliminate the need to operate the low flow bypass gate at openings less than 6 inches. Elliptical curves are provided on intake passage roof and sides. A trashrack will be placed at the entrance of the 5-foot by 7-foot wet well sluice gate conduit to prevent trash entering the minimum discharge line from the main well. Trashrack openings are 6-inch square; this will have to be refined at the FDM level when the final type of valving is selected. Provisions have been made for upstream maintenance bulkheading. The design condition for minimum discharge line is to control lower flows which are required by the operation schedule (see figures 6-19 and 6-20). This line will consist of a 3.25-foot-diameter steel pipe originating at the bottom of the multilevel withdrawal drop well. Flow will be controlled with a 14-inch disk gate valve. A ball valve has been provided upstream of this gate valve to shut off flow in case the disk gate valve requires maintenance. The minimum discharge line can be operated with the wet well sluice gate into the main wet well closed, allowing releases to be made while the main wet well is dewatered for inspection or repair. The rating curve for the minimum discharge line is shown on figure 6-22. The loss coefficient K values used for design for operation of the minimum discharge line are summarized in table 6-11. An entrance loss of 0.10 was used relative to the intake passage for rating calculations. An absolute roughness value of 0.00015 feet was used, based on figure 10-9, page 377, "Engineering Fluid Mechanics," Roberson/Crowe. A discharge line length of 1,680 feet was used for computations. Gate valve losses for various gate openings were determined using Hydraulic Design Criteria Chart 330-1/1. Cavitation may occur at partial gate openings and will be addressed at the FDM level. To size the line (ensure adequate capacity), an absolute roughness value of 0.003 feet was used. This corresponds to heavy rust. Head loss through the low pool multilevel withdrawal system has been accounted for as described in paragraph 6.6.e(3). Air supply to minimum discharge line and/or alternative gate valve types should be further investigated at FDM level.

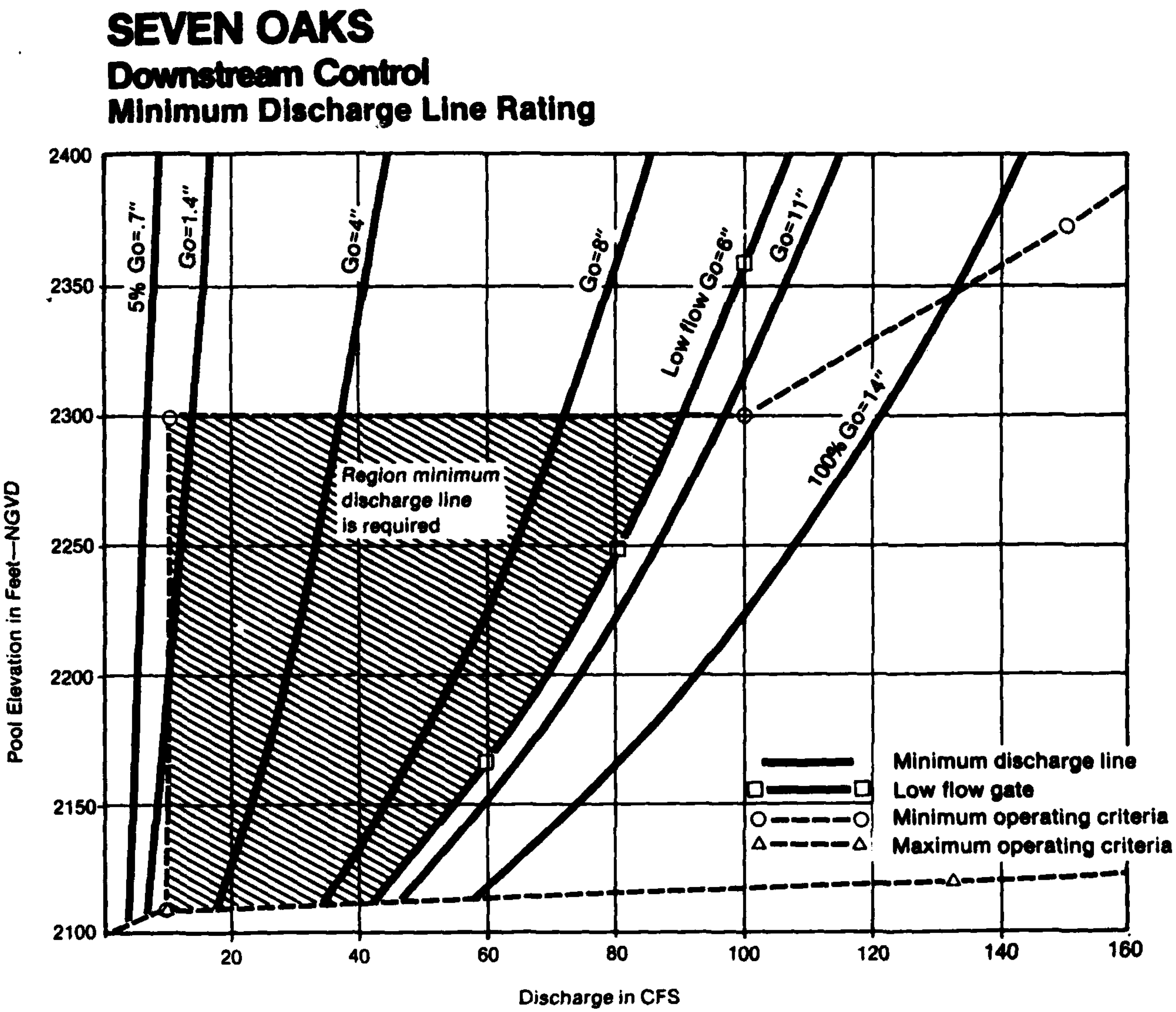


Figure 6-22 Minimum discharge line rating curves, 14" Gate Valve.

Table 6-11. Downstream Control - Energy Loss Coefficients* (K Values) for Minimum Discharge Line

	RATING
Intake Entrance	0.001 (1)
U/S Transition	---- (2)
U/S Bends	0.005 (3)
Tunnel Friction	0.093 (4)
D/S Bends	0.003 (5)
D/S Transition	0.02 (6)
Gate Slot	0.01 (7)
Total	0.132

* Note that all loss coefficients are referenced to the conduit proper upstream of the gate.

(1) "Handbook of Hydraulics," Brater and King, page 6-21.

(2) "EM 1110-2-1602," plate C-9. Negligible.

(3) "Handbook of Hydraulics," page 6-24, 25.

(4) "Handbook of Hydraulics," page 6-12 and "Engineering Fluid Mechanics," Roberson/Crowe, page 376, absolute roughness = 0.00015 feet, f = .0106, D = 3.25 feet, L = 1,680 feet.

(5) Same as (3).

(6) "EM 1110-2-1602," plate C-9.

(7) "EM 1110-2-1602," paragraph 3-7, page 3-5.

g. Aeration Scheme. Twelve-inch floor offsets and 6-inch wall offsets will be located 4.5 feet downstream of service gates for main intakes (see plates 3-6 and 3-7). A 6-inch floor offset and a 6-inch wall offset will be used for the low flow bypass. Offsets have been selected to ensure that air is insufflated into flow along boundaries. The aerated boundary will act as a cushion and provide protection to keep vapor cavities from collapsing against concrete surfaces. Offsets were selected over air slots because: offsets will not fill with water as air slots can; offsets will not fill with sediment; offsets provide more water surface for aeration; offsets separate flow surfaces from jet for longer distances, entraining more air; offsets are less critical to construct; and offsets increase the height and width of the conduit which is favorable in transitioning from the gate passages to the RO exit chute. Preliminary design of offsets was made using recommendations in "Hydraulic Model Studies of Chute Offsets, Air Slots, and Deflectors for High-Velocity Jets," REC-ERC-73-5, G. L. Beichley, Bureau of Reclamation, March 1973. The 14-inch circular minimum discharge line will exit directly into a rectangular section, 3 feet wide, downstream of the disk gate valve. This will provide a minimum 6-inch floor offset and minimum of 11-inch wall offsets. Note that aeration scheme for minimum discharge line should be investigated in more detail at FDM level.

h. Air Demand.

(1) General. Air requirements were estimated using the four different methods listed below:

(a) EM 1110-2-1602 design guidance.

(b) Bureau of Reclamation, Engineering Research Center, computer program on aeration.

(c) Assumed velocity distribution for air in RO conduit.

(d) Libby Dam model and prototype test results.

Results from the four methods were compared and used to estimate the total air demand, where total air demand is a combination of: air entrained in water; and air moving above water surface (surface air demand) due to shear field created by relative movement of water and air.

(2) Main Intake Conduits. The maximum air demand is expected to fall in the range of 3,200 to 5,200 cfs per main conduit. The upper limit was used for design. This gives a maximum ratio of air demand to water discharge of 1.3. Air will be supplied through a rectangular box located on top of the crane deck just downstream of the gate control room. The rectangular box is 9 feet wide by 37 feet long by 5 feet high. The air passage system will be located approximately 4 feet downstream from the offsets in the main conduit. Air will be supplied through a 9-foot by 37-foot grating on the top of the box and two 5-foot by 9-foot gratings on the sides, as shown in plate 3-7. The gratings are provided for safety and to prevent undesirable debris from being drawn in. Velocities into the air intake will be less than 30 fps. The air distribution through the intake should be further analyzed at the FDM level. The only head loss through the air passage system will occur at the intake. The maximum head loss should be less than 1.0 foot of water, the limit recommended in general EM guidance. This will be studied in more detail for the FDM.

(3) Low Flow Bypass. Five hundred cfs is the largest discharge which will have to be passed through the low flow bypass (see figure 6-20). Based on results from the four methods listed in paragraph 6.6.h(1), the maximum air demand might be as high as 1.3 times the maximum water discharge. The validity of this factor for the low flow bypass should be checked at the FDM level, since the factor of 1.3 is based on flow through the main RO conduit. An air demand of 650 cfs was used for design. The same air supply system will be used for the main RO conduit, low flow bypass, and minimum discharge line. There will be sufficient air available for low flow bypass and minimum discharge line, and both air velocities and head loss through air supply will be acceptable, since the air supply system was sized based on the larger air demand required for the main RO conduit.

i. Regulating Outlet Exit Chute. The regulating outlet chute is shown on plates 3-5 and 3-6. Each of the RO gate passages, 5 feet wide by 9 feet high, transition into an 11-foot wide by 16-foot high exit chute. The transition takes place over a distance of 53 feet beginning at the offset. Four and one-half feet downstream of the RO gates, the section expands via roof expansion, a 1-foot offset on the floor, and 0.5-foot offset on each wall. The conduit floor and walls will be steel lined from the RO gates downstream to the offsets. The 6-foot-wide section expands to 11 feet at a slope of one on twenty-one. The increase in width is necessary to reduce the unit flow rate for energy dissipation. Also, by increasing the width, the distance from the offset at which the jet impacts the exit chute is greater, entraining more air in the flow. The conduit height increases to 19 feet, 4 feet downstream of offsets. The exit chute will be covered for a distance of 57.5 feet downstream of the RO service gates, except for the 9-foot-wide air supply intake. The roof should prevent significant spray caused by turbulence of flow at the gates and offsets, and should prevent overtopping of the exit chute walls. The roof will also be used as a crane deck. The roof will end at the same station as the transition, Station 28+59, and the wall heights decrease to 16 feet. The flow profile will be more uniform downstream of the gates, and the roof and higher walls should no longer be required to contain spray. Exit chutes for the minimum discharge line and low flow bypass have a constant width of 3.0 feet downstream of the offsets. Wall heights are the same as for the main RO exit chutes except downstream of Station 28+59, where the wall heights decrease to 8 feet for the minimum discharge exit chute. Maximum velocities through the system were estimated for both design discharges and maximum gate openings using minimum loss coefficients (see paragraph 6.6.e). Results are summarized in table 6-12 for velocities at the vena contracta and velocities in the exit channel at Station 30+40. Note that velocity computations for the exit channel assumed no loss due to the offsets, no loss due to expansion, and neglected effects of air entrainment. A Manning's "n" value of 0.008 was used to estimate the flow profile through the exit chutes. Since there are separate conduits and exit chutes for the RO, low flow bypass, and minimum discharge line, calculations for each of these systems were made independently. Maximum depths were also estimated using capacity loss coefficients and results are summarized in table 6-13. An energy loss of 25 percent was used for the main RO chutes to account for

Table 6-12. Downstream Control - Maximum Velocities in Exit Chute

Pool El. (feet-NGVD)	System	Percent Gate Opening	Total Discharge (cfs)	Max. Velocity (fps) Vena Contracta	Max. Velocity (fps) Station 30+40	Min. Depth (feet) Station 30+40
2,580	RO*	72	8,000	160	145	2.5
2,580	RO*	100	11,440	130	120	4.3
2,580	RO**	66	4,000	175	160	2.3
2,580	RO**	100	7,022	155	150	4.3
2,580	Low Flow	66	500	140	100	1.7
2,580	Low Flow	100	715	100	80	2.9
2,300	Min. Discharge	73	90	125	45	0.65
2,580	Min. Discharge	100	180	175	80	0.75

* Both gates operating at equal openings.

**One gate operating.

Table 6-13. Downstream Control - Maximum Depths in Exit Chute

Pool El. (feet-NGVD)	System	Percent Gate Opening	Total Discharge (cfs)	Minimum Velocity (fps) Vena Contracta	Minimum Velocity (fps) Station 30+40 0% Bulk	Maximum Depth (feet) Station 30+40 0% Bulk	Maximum Depth (feet) Station 30+40 50% Bulk	Maximum Depth (feet) Station 30+40 130% Bulk
2,580	RO*	95	8,000	110	80	4.6	6.9	10.6
2,580	RO*	100	8,615	95	70	5.7	8.6	13.1
2,580	RO**	70	4,000	160	110	3.3	5.0	7.6
2,580	RO**	100	5,850	130	95	5.6	8.4	12.9
2,580	Low Flow	100	500	71	30	5.5	8.3	12.7
2,580	Low Flow	100	510	73	31	5.5	8.3	12.7
2,300	Min. Dis.	75	90	120	16	1.9	2.9	4.4
2,580	Min. Dis.	100	170	170	30	1.9	2.9	4.4

* Both gates operating at equal openings.

**One gate operating.

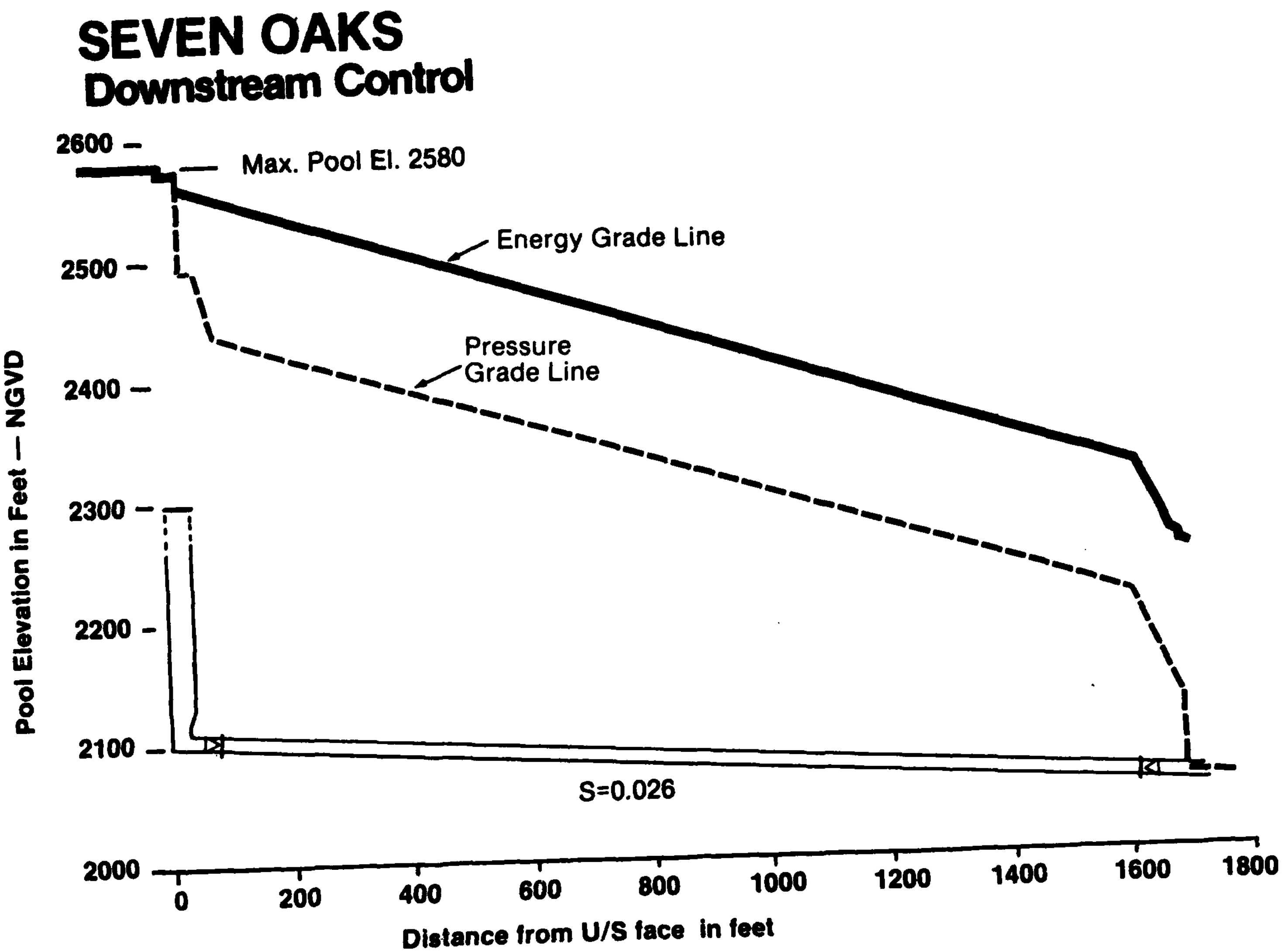


Figure 6-23 EGL and PGL for Regulating Outlet Works, Q=8000 cfs

losses at the offset, transition, and frictional loss through the transition. Energy losses of 15 and 20 percent were assumed for losses at the offset, for the low flow bypass, and minimum discharge line, respectively. A Manning's "n" value of 0.015 was used to estimate the flow profile through the exit chutes for capacity calculations. To estimate the effects of air entrainment on flow depths, depths were calculated with 50 percent and 130 percent bulking. A hydraulic model study should be conducted at the FDM level to confirm that the entire exit chute transition will be hydraulically acceptable because of high flow velocities and corresponding cavitation potential and difficulty in accurately defining flow profiles associated with offsets.

j. Energy Grade Line and Pressure Grade Line. The energy grade line (EGL) and pressure grade line (PGL) have been computed from the intake downstream to the RO gates for a discharge of 8,000 cfs through the RO works. For pressure flow downstream to the gates, the velocity head in any section remains constant for a fixed discharge. Minimum pressures and maximum gate opening will occur for maximum head loss through the system. Therefore, maximum loss coefficients (see paragraph 6.6.e) were used for the EGL and PGL shown in figure 6.23.

6.7 Mid-Tunnel Control.

a. General. The mid-tunnel control alternative was analyzed for reasons discussed in paragraph 1.7. Advantages and disadvantages are listed in section 5. Details of the analysis provided are described in the following text.

b. Regulating Outlet Intake. The RO entrance invert will be at El. 2,100 feet. An 18-foot-diameter horseshoe shaped intake will be used. Provisions have been made for maintenance bulkheading. The entrance curves will have 2-foot radii. Elliptical entrance curves are not required because the intake area is large, 289 square feet. The average flow velocity at the entrance will be 28 feet per second for the design discharge of 8,000 cubic feet per second. Positive pressures can be maintained at the intake for all flows so entrance curves are adequate as stated in EM 1110-2-1602, paragraph 3-6. The side and roof curves will be the same. The intake has been extended into the wet well to provide an acceptable transition from the circular well to a flat front entrance.

c. Pressure Conduit. An 18-foot-diameter horseshoe conduit has been designed to operate under pressure flow. At the design discharge of 8,000 cfs the average velocity in the concrete conduit will be 27.7 fps. The smooth pipe curve from the Moody diagram was used to calculate friction loss in the pressure conduit for maximum velocities. Absolute roughness values for rating and capacity computations are .001 and 003 feet, respectively, and were based on a concrete surface in good condition and unusually rough (see "Engineering Fluid Mechanics," by Roberson and Crowe, and "Handbook of Hydraulics," by Brater and King). Friction values, f, obtained from the Moody diagram are .009, .011, and .013 for velocity, rating, and capacity computations, respectively. The frictional loss coefficients, K, to be used in the Darcy-Weisbach equation were calculated to be 0.17, 0.23, and 0.27, relative to the 18-foot-diameter horseshoe conduit and a length of 363 feet.

d. Mid-Tunnel Transition. The transition from the 18-foot-diameter horseshoe conduit to the main RO and low flow conduit intakes occur over a distance of 25 feet (see plate 4-5). Approximately 365 feet downstream of the 36-foot-diameter wet well, the 18-foot-diameter horseshoe conduit transitions into the 18-foot-high by 23-foot-wide section over a distance of 25 feet. The transition length was calculated for an offset distance of 2.5 feet and is based on design guidance in EM 1110-2-1602, page 4-13. The angle of expansion relative to the conduit centerline is 5.7 degrees. Downstream of the transition the rectangular conduit remains constant in cross section for 25 feet. At the end of this rectangular section, two 5.5-foot-wide piers symmetrically split the flow into two 5-foot-wide by 9-foot-high main RO conduits and a 2-foot-wide by 3.5-foot-high low flow bypass conduit. Each of the main RO conduit intakes is sized to pass approximately 75 percent of the design discharge at pool El.2,580, with only one of the gates open. Conventional one on three elliptical curves have been selected for the roof and inner side curves of the entrance to the gate passages and these curves form the pier end curves. The service gates are approximately 53 feet downstream from the start of the piers. The invert of the gates is at El. 2,090.2. An upstream emergency gate will be provided in each passage. A loss coefficient (minor loss) through the transition of .1, .25, and .4 relative to the area of the gate passages was used for velocity, rating, and capacity

design, respectively. The smooth pipe curve from the Moody diagram was used to calculate friction loss for maximum velocities. Absolute roughness values for rating and capacity computations are .001 and .003 feet. A hydraulic model study should be conducted at the FDM level to confirm that the entire transition will be hydraulically acceptable.

e. Regulating Outlet Gate Rating.

(1) General. The RO rating curves for various gate openings are shown on figure 6-24 (high level intake) and figure 6-25 (multilevel withdrawal system). Figure 6-26 shows regions where minimum discharge line, low flow bypass, and main regulating outlet are required when multilevel withdrawal system is used. Average values of loss coefficients have been used to approximate actual operating conditions. Minimum, average, and maximum loss coefficients have been calculated from the pool to the regulating gates and were used for velocity, rating curve, and capacity design, respectively.

(2) Energy Loss Coefficients for High Level Intake. Average loss coefficients for the high level intake have been computed and are summarized in table 6-14. Trash strut loss coefficients have been determined for velocity, capacity, and rating, and are described in paragraph 6.2.b. Wet well friction loss coefficients were calculated, but were found to be negligible as stated in paragraph 6.3.b. RO intake losses are as shown in table 6-14. The smooth pipe curve and absolute roughness values of .001 and .003 feet were used to compute friction "f" values for the pressure conduit as described in paragraph 6.7.c. Minor loss coefficients due to the transition from the pressure conduit to separate gate passages are described in paragraph 6.7.d. The total loss coefficients for velocity, rating, and capacity are shown in table 6-14. All loss coefficients are relative to the velocity through the gate passages.

(3) Energy Loss Through Multilevel Withdrawal System. To develop the rating curves for the RO with flow entering the wet well through the multilevel withdrawal system, the head loss in feet through trashracks, ports, multilevel drop well, and wet well sluice gate passage was

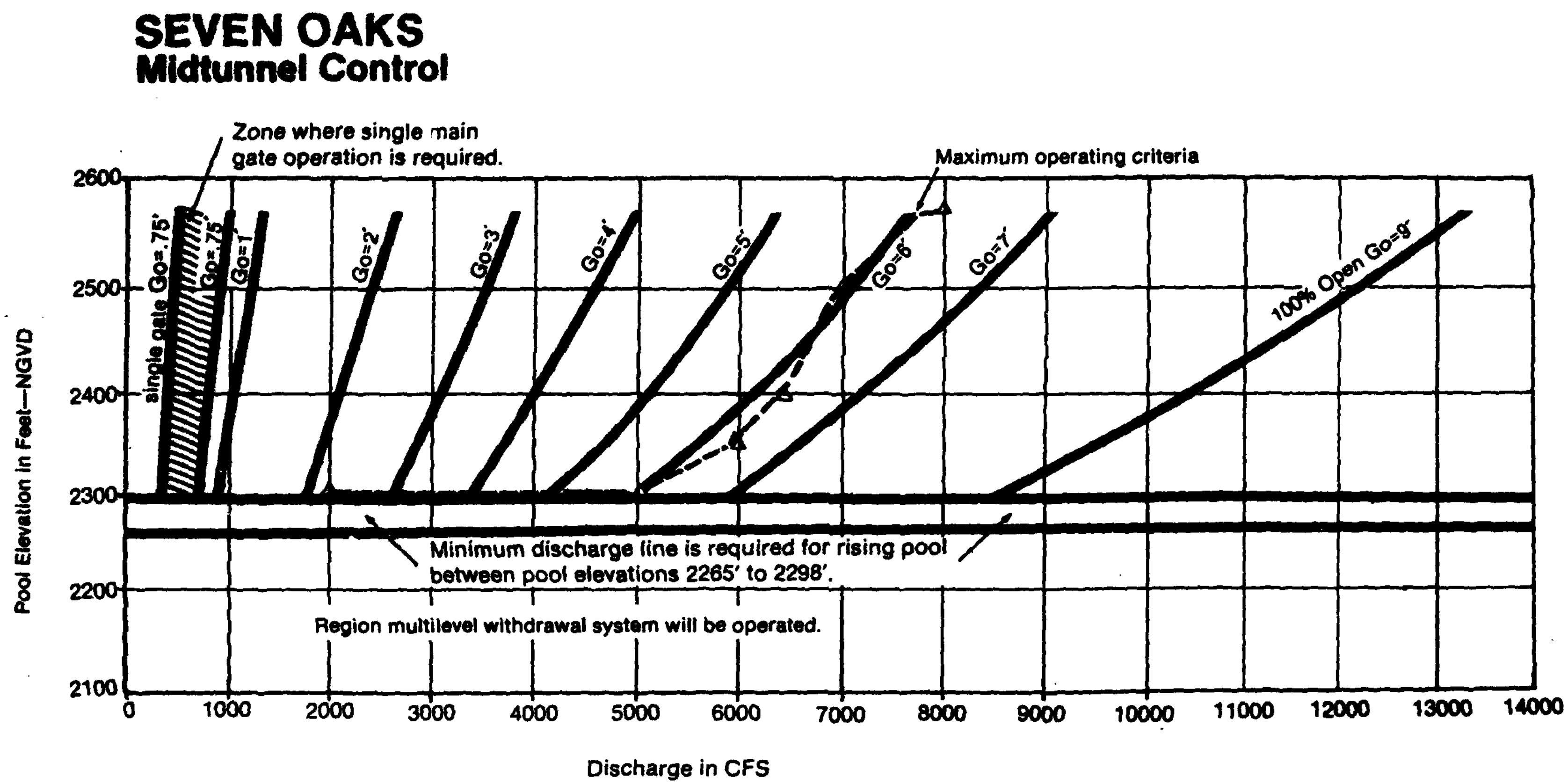


Figure 6-24 High level intake-Main gate rating, 2 gates operating with balanced operation.

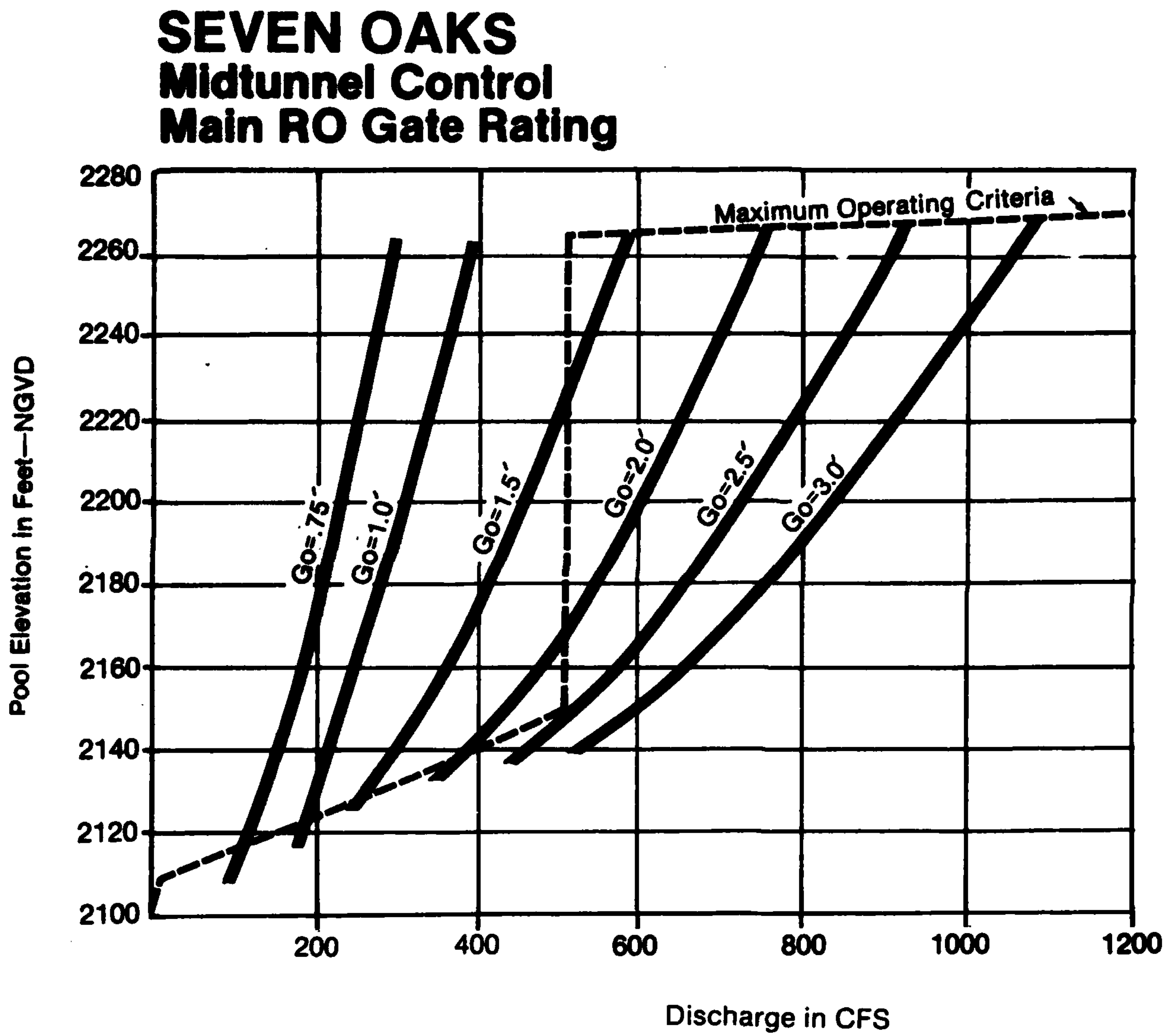


Figure 6-25 Multilevel withdrawal system—rating curve for a single main operating gate.

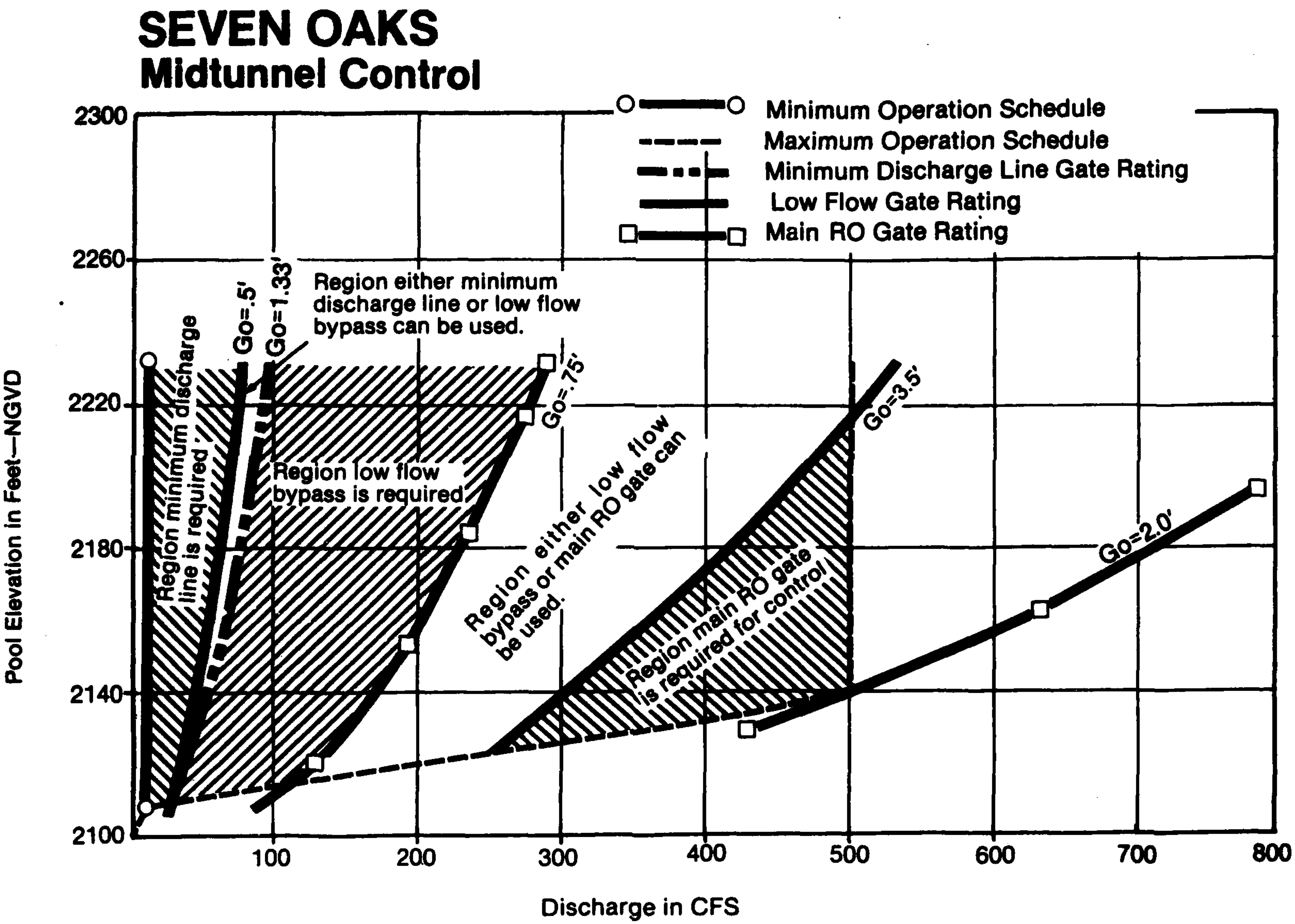


Figure 6-26 Multilevel withdrawal system—Regions minimum discharge line, low flow bypass, and main regulating outlets will be required.

Table 6-14. Mid-Tunnel Control - Energy Loss Coefficients* (K values) for High Level Intake and Main RO Gates

	Capacity	Rating	Velocity
Trash struts	0.028 (1)	0.011(2)	0.005 (3)
Drop well	----- (4)	---- (5)	----- (6)
RO intake	0.039 (7)	0.029(8)	0.019 (9)
Bulkhead slot	----- (10)	-----	-----
Friction - pressure conduit, transition, and gate passages	0.138 (11)	0.113(12)	0.069(13)
Mid-tunnel transition	0.4 (14)	0.25	0.10
Emergency gate slot	0.01 (15)	0.01	0.01
Total	0.615	0.413	0.203

*Note that all loss coefficients are referenced to conduit proper upstream of gates and are for both gates being operated equally.

(1) "Design of Small Dams," Bureau of Reclamation, page 366, eq. 11, trash struts 50 percent clogged.
(2) Same reference as for 1, trash struts 25 percent clogged.
(3) Same reference as for 1, trash struts 0 percent clogged.
(4) Darcy-Weisbach equation - equivalent f, given Manning's roughness coefficient n = 0.015. Found to be negligible, less than 0.0005.
(5) Same as for 4 except n = 0.012. Found to be negligible, less than 0.0005.
(6) Same as for 4 except n = 0.008. Found to be negligible, less than 0.0005.
(7) Entrance loss of 0.4 times velocity head in 18-foot horseshoe section. Note that for a square edged entrance, the coefficient would be 0.5, "Handbook of Hydraulics," Brater and King, pg 6-20.
(8) Entrance loss of 0.3 times velocity head in 18-foot horseshoe section, reference EM 1110-2-1608," paragraph 3-7, page 3-5, and Lost Creek Dam Computations.
(9) Entrance loss of 0.2 times velocity head in 18-foot horseshoe section.
(10) EM 1110-2-1602, paragraph 3-7, page 3-5.
(11) EM 1110-2-1602, paragraph 2-12.g. (1)(a), page 2-10, e = .003.
(12) "Handbook of Hydraulics," Brater and King, pages 6-12, and "Engineering Fluid Mechanics," page 376, e = .001.
(13) EM 1110-2-1602, paragraph 2.12.g.(1)(b), page 2-10, smooth pipe curve.
(14) Hydraulic Design Chart, 221-1/3.
(15) EM 1110-2-1602, paragraph 3-7, page 3-5.

calculated for various discharges. The rating curve for the high level intake was then adjusted by adding the head loss to the pool elevation for the corresponding discharge. This adjustment was possible since losses through the high level trash struts and main wet well are negligible for the range of discharges passing through the multilevel withdrawal system. The trashrack loss coefficient value of 1.11 for the multilevel withdrawal system, referenced to velocity head through the net area of a trashrack for one row of ports, was determined for 25 percent clogging using equation 11, page 366, "Design of Small Dams," Bureau of Reclamation. An entrance loss coefficient K value of .98 was used for each port. This value was calculated by converting the average (.71) of discharge coefficients for a short tube (.82) and sharp edged orifice (.60) to an entrance loss coefficient. References used were "Handbook of Hydraulics," by Brater and King, page 4-19 through page 4-35, and "Design of Small Dams," Bureau of Reclamation, page 363. An exit loss coefficient K value of 1.0 was assumed for flow exiting ports into the multilevel wet wells. Friction loss coefficient, K value, in the multilevel wet well was estimated using an equivalent Manning's "n" value of .012. Entrance loss into the passage between wet wells, referenced to the velocity head in passageway, was estimated at .66. This value was based on coefficients of discharge for submerged tubes with square cornered entrances found in Kings Handbook, table 4-35, page 4-35, and "Design of Small Dams," page 363. Friction in the passage was found to be negligible because of the short passage length. For flow exiting the wet well sluice gate passage into the main drop well, an exit loss coefficient K value of 1.0 was assumed. Losses at the RO entrance and downstream to the vertical slide gates are the same as described in paragraph 6.8.b and listed on table 6-14.

f. Low Flow.

(1) Low Flow Bypass. The low flow bypass shown on plates 4-5 and 4-6 will eliminate the need to operate RO slide gates at openings of less than 9 inches. The entrance invert will be at El. 2,090.2. Flow will pass through the 18-foot horseshoe conduit until it reaches the intake to the low flow bypass gate passage. Elliptical curves are provided on the intake passage

roof and sides. A semi-circular trashrack, 18 feet high with a 4.5-foot radius, will be provided at the upstream end of the intake piers. The trashrack will prevent material which is small enough to pass through the trash struts for the high level intake from blocking the low flow slide gate. Maintenance of this trashrack will be an important consideration at the FDM level. Flow will be controlled with a 2-foot-wide by 3.5-foot-high vertical slide gate. The low flow bypass rating curves are shown on figure 6-27, high level intake, and figure 6-28, low pool multilevel withdrawal system. Figure 6-27 shows: minimum required flow conditions for Seven Oaks operation; discharge rating curves for a single 5-foot-wide by 9-foot-high main RO gate with a 9-inch opening; and discharge rating curves for the RO low flow bypass conduit. The design condition for low flow bypass is discharges less than those which can be controlled with a main gate opening of 9 inches, but those discharges which are required by the operation schedule. When the multilevel withdrawal system is being operated, the design discharge through the low flow bypass will be 290 cfs and occur at pool El. 2,265 (at pool El. 2,265, 290 cfs can be controlled with a main gate opening of 9 inches) as shown in figure 6-26. The bottom of the high level intake is at El. 2,265. When the high level intake is being used, the design discharge through the low flow bypass will be 485 cfs and occur at pool El. 2,580 (at pool El. 2,580, 485 cfs can be controlled with a main gate opening of 9 inches) as shown in figure 6-27. The low flow bypass has been sized for more than minimum capacity of the main gates, so there are regions where either the low flow gate or main gate can be used to control flow, as shown in figures 6-26 and 6-27. The energy loss coefficient K value, used for design for operation of the high level intake are summarized in table 6-15. The average loss coefficient between the pool and the section just upstream of the gate, for the high level intake and relative to the area of low flow bypass intake conduit cross section, is .512. Head loss through the low pool multilevel withdrawal system has been accounted for as described in paragraph 6.7.e(3).

SEVEN OAKS
Midtunnel Control

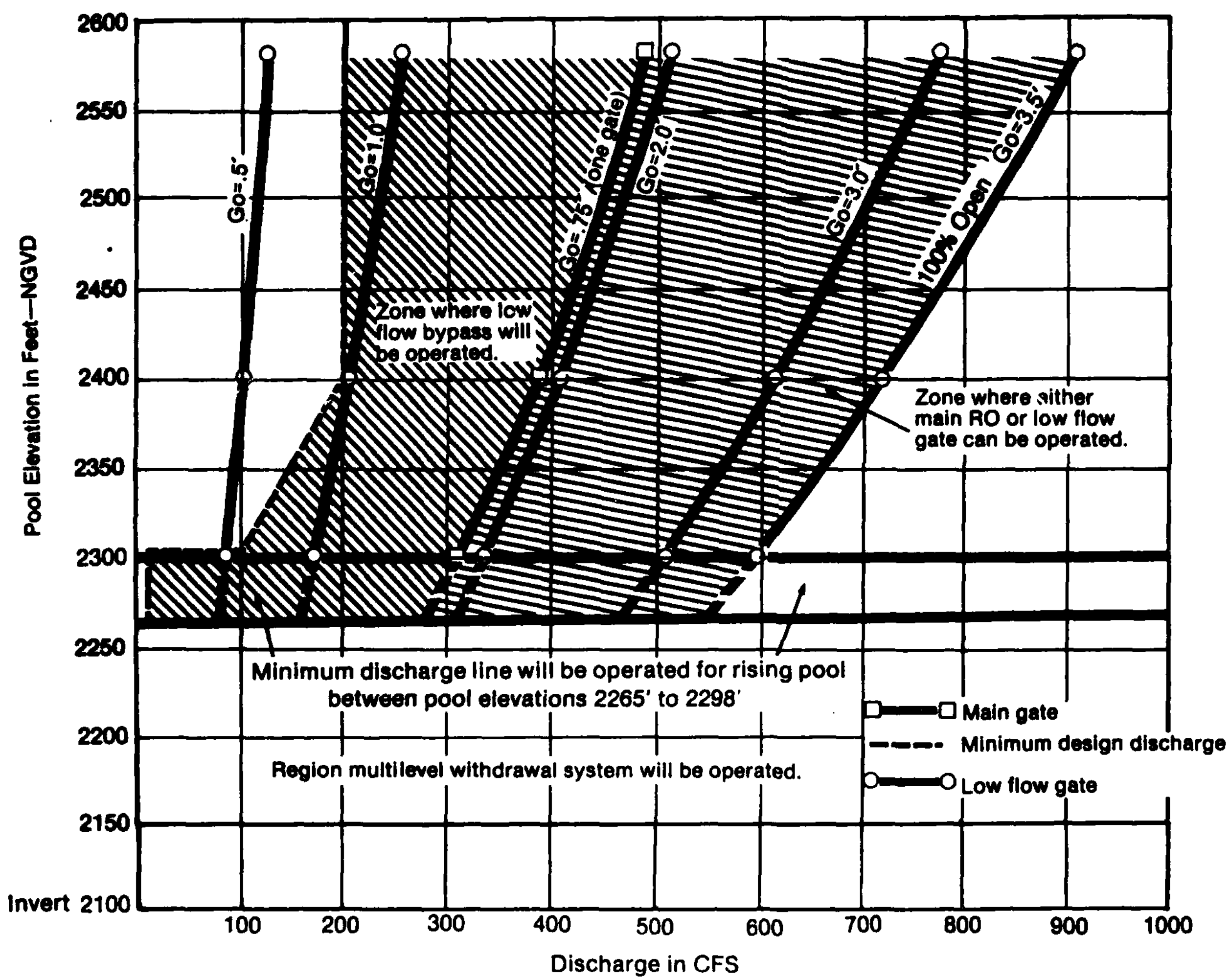


Figure 6-27 High level intake—rating curve for low flow bypass.

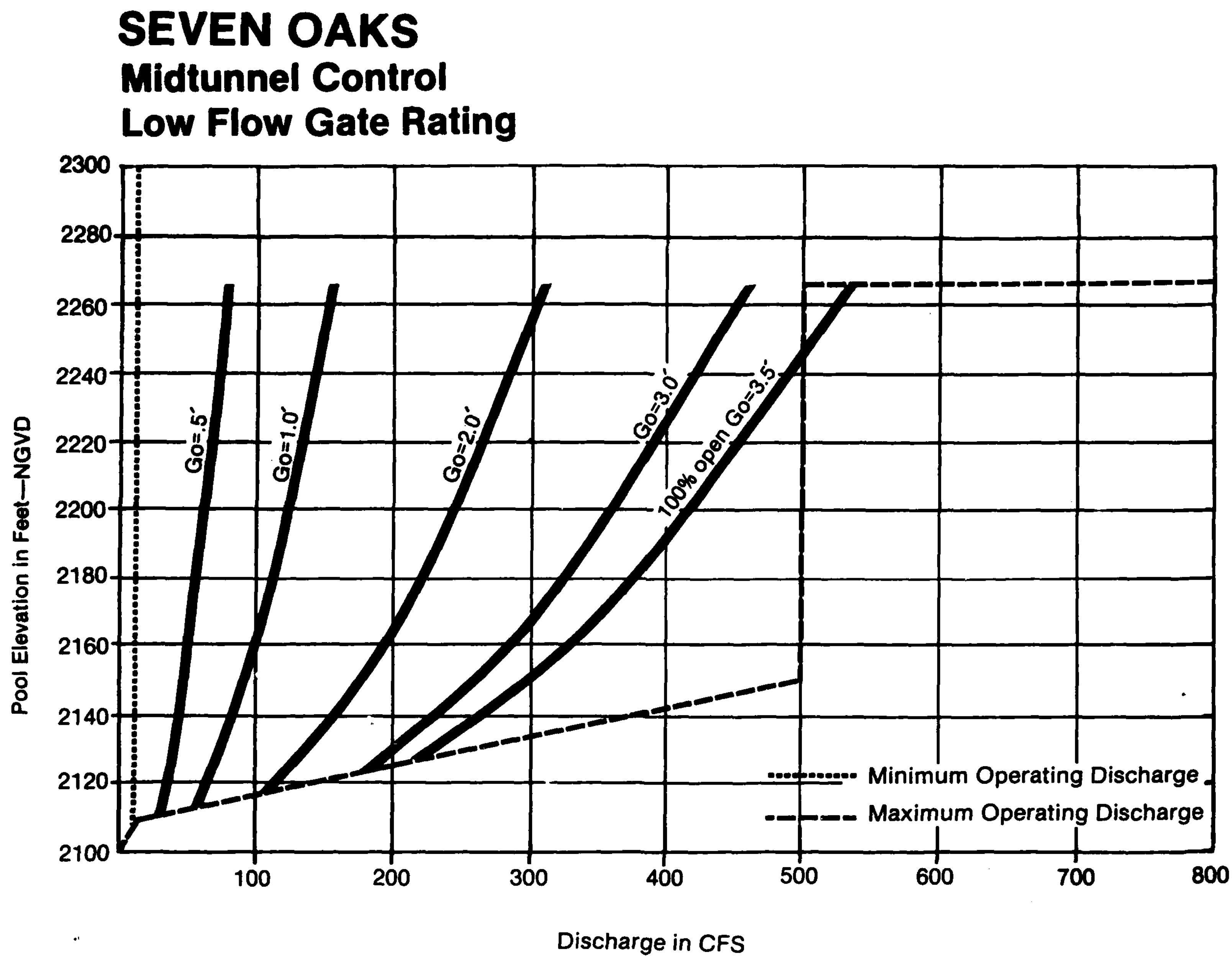


Figure 6-28 Multilevel withdrawal system—rating curve for low flow bypass

Table 6-15. Mid-Tunnel Control - Energy Loss Coefficients* (K Values) for High Level Intake and Low Flow Bypass

	RATING
Trash struts	---- (1)
Wet well	---- (2)
Bulkhead slot	---- (3)
Intake entrance	---- (4)
Friction - pressure conduit, transition, and gate passages	.252 (5)
Mid-tunnel transition	.250 (6)
Gate Slot	.010 (7)
Total	.512

*Note that all loss coefficients are referenced to conduit proper upstream of gates.

(1) "Design of Small Dams," Bureau of Reclamation, page 366, eq. 11, trash struts 25 percent clogged. Negligible.

(2) Darcy-Weisbach equation - equivalent f, given absolute roughness value E = .001 feet. Negligible.

(3) EM 1110-2-1602, paragraph 3-7, page. 3-5. Negligible.

(4) Entrance loss relative to 18-foot horseshoe section. Reference "EM 1110-2-1602," page 3-5, and Lost Creek Dam computations. Negligible.

(5) "Handbook of Hydraulics," page 6-12, and "Engineering Fluid Mechanics," Roberson/Crowe, page 376, absolute roughness = .001 feet, f = .016, D = 2.54 feet, L = 40 feet. Friction in 18-foot horseshoe tunnel was negligible. Friction loss coefficient is for loss in the gate passage.

(6) "Hydraulic Design Chart," 221-1/3.

(7) EM 1110-2-1602, paragraph 3-7, page 3-5.

(2) Minimum Discharge Line. The minimum discharge line shown on plates 4-5 and 4-6 will eliminate the need to operate the low flow bypass gate at openings less than 6 inches. Elliptical curves are provided on intake passage roof and sides. A trashrack will be placed at the entrance of the 5-foot by 7-foot wet well sluice gate conduit to prevent trash entering the main well from reaching the minimum discharge line. Trashrack openings are 6-inch square; this will have to be refined at the FDM level when the final type of valving is selected. Provisions have been made for upstream maintenance bulkheading. The design condition for minimum discharge line is to control lower flows which are required by the operation schedule (see figures 6-26 and 6-27). This line will consist of a 2-foot-diameter concrete pipe originating at the bottom of the multilevel withdrawal drop well. Flow will be controlled with a 16-inch disk gate valve. The invert of disk gate valve is at El. 2,102.2. An upstream emergency gate valve has been provided. The minimum discharge line can be operated with the wet well sluice gate closed, allowing releases to be made while the main wet well is dewatered for inspection or repair. The rating curve for the minimum discharge line is shown on figure 6-29. The loss coefficient K values used for design for operation of the minimum discharge line are summarized in table 6-16. An entrance loss of 0.10 was used relative to the intake passage for rating calculations. An absolute roughness value of 0.001 feet was used, based on figure 10-9, page 377, "Engineering Fluid Mechanics," Roberson/Crowe. A discharge line length of 440 feet was used for computations. Gate valve losses for various gate openings were determined using Hydraulic Design Criteria Chart 330-1/1. Cavitation may occur at partial gate openings and will be addressed at the FDM level. To size the line (ensure adequate capacity) an absolute roughness value of 0.003 feet was used. Head loss through the low pool multilevel withdrawal system has been accounted for as described in paragraph 6.7.e(3). Air supply to minimum discharge line and/or alternative gate valve types should be further investigated at FDM level. An alternative plan for the minimum discharge line is a 3-foot-diameter pipe that will carry flow from the multilevel withdrawal well to the end of the RO works which will be regulated at the downstream end. Further design for this alternative will be done at the feature design level.

Seven Oaks
Midtunnel Control
Minimum Discharge Line Rating

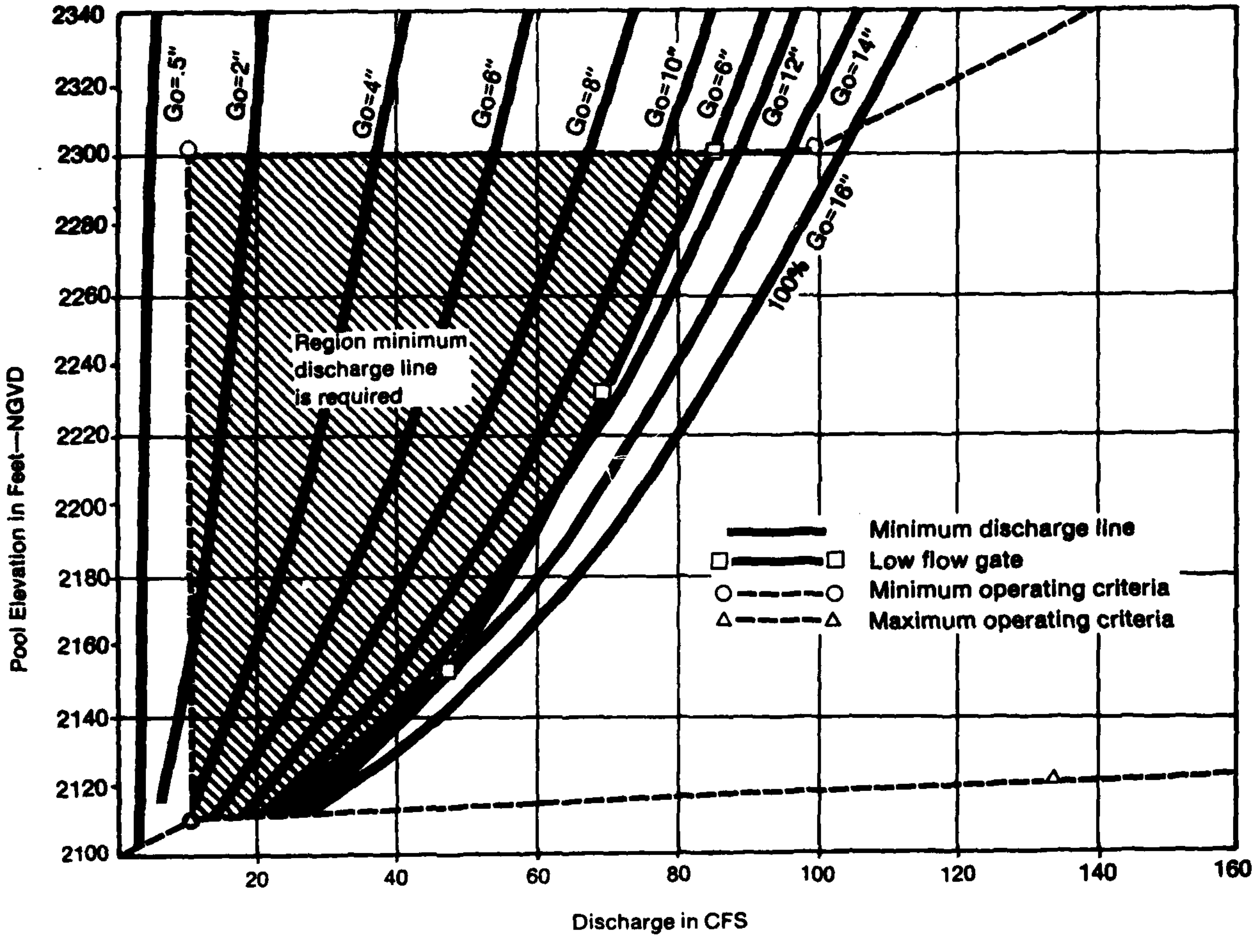


Figure 6-29 Minimum discharge line rating curves, **16" Gate Valve.**

Table 6-16. Mid-Tunnel Control - Energy Loss Coefficients* (K Values) for Minimum Discharge Line

	RATING
Intake entrance	.020(1)
U/S bulkhead slot	.002(2)
Bends	.166(3)
Contractions	.006(4)
Friction	1.031(5)
D/S gate slot	.010(6)
---------------	-----------
Total	1.235

*Note that all loss coefficients are referenced to the conduit proper upstream of the gates.

(1) "Handbook of Hydraulics," Brater and King, page 6-21.

(2) EM 1110-2-1602, paragraph 3-7, page 3-5.

(3) "Handbook of Hydraulics," page 6-24, 25.

(4) "Engineering Fluid Mechanics," Roberson/Crowe, p 384.

(5) "Handbook of Hydraulics," page 6-12, and "Engineering Fluid Mechanics," page 376, absolute roughness = .001 feet.

(6) Same as (2).

g. <u>Aeration Scheme</u>. Twelve-inch floor offsets and 6-inch wall offsets will be located 4.5 feet downstream of service gates for main intakes (see plates 4-5 and 4-6). A 6-inch floor offset and a 6-inch wall offset will be used for the low flow bypass. Offsets have been selected to ensure that air is insufflated into flow along boundaries. The aerated boundary will act as a cushion and provide protection to keep vapor cavities from collapsing against concrete surfaces. Offsets were selected over air slots because: offsets will not fill with water as air slots can; offsets will not fill with sediment; offsets provide more water surface for aeration; offsets separate flow surfaces from jet for longer distances, entraining more air; offsets are less critical to construct; and offsets increase the height and width of the conduit which is favorable in transitioning from the gate passages to the main tunnel. Preliminary design of offsets was made using recommendations in "Hydraulic Model Studies of Chute Offsets, Air Slots, and Deflectors for High-Velocity Jets," REC-ERC-73-5, G. L. Beichley, Bureau of Reclamation, March 1973. The circular minimum discharge line will exit directly from the 16-inch disk gate valve into a rectangular section, 3 feet wide, downstream of the disk gate valve. This will provide a minimum 4-inch floor offset and minimum of 10-inch wall offsets. Note that aeration scheme for minimum discharge line should be investigated in more detail at FDM level.

h. <u>Air Demand</u>.

(1) <u>General</u>. Air requirements were estimated using the four different methods listed below:

(a) EM 1110-2-1602 design guidance.

(b) Bureau of Reclamation, Engineering Research Center, computer program on aeration.

(c) Assumed velocity distribution for air in RO conduit.

(d) Libby Dam model and prototype test results.

Results from the four methods were compared and used to estimate the total air demand, where total air demand is a combination of: air entrained in water; and air moving above water surface (surface air demand) due to shear field created by relative movement of water and air.

(2) Main Intake Conduits. The maximum air demand is expected to fall in the range of 3,200 to 5,200 cfs per main intake conduit. The upper limit was used for design. This gives a maximum ratio, of air demand to water discharge, of 1.3. Air will be supplied to the main RO gates by a 10-foot-diameter vertical shaft located adjacent to the mid-tunnel tower as shown on plates 4-5 and 4-6. The air passage has an area of 78.5 square feet. Maximum velocity through the air supply shaft will be 135 fps, less than the maximum of 150 fps recommended in EM 1110-2-1602. The intake for the air supply has been designed to ensure that velocities are less than 30 fps, as recommended in EM 1110-2-1602, paragraph 3.17.d. The one main vertical shaft manifolds into two 6-foot by 6-foot passageways to vent each of the gates. The air vents terminate into 6-foot by 6-foot plenums located in the conduit roofs directly above the wall offsets. Maximum headloss through the air passage system was calculated to range between 1.5 and 2.4 feet of water which is higher than recommended in the EM. The maximum head loss through air passage system was calculated using conservative loss coefficients and high air demand. If care is used in design of bends, transitions, etc., at FDM level, it should be possible to reduce head loss. In this case, due to large costs associated with the size of air vents, an exception was made to the general EM guidance.

(3) Low Flow Bypass. Four hundred eight-six cfs of water is the largest discharge which will have to be passed through the low flow bypass (see figure 6-27). Based on results from the four methods listed above, the maximum air demand might be as high as 1.3 times the maximum water discharge. The validity of this factor for the low flow bypass should be checked at the feature design level, since the factor of 1.3 is based on flow through the main RO conduit. An air demand of 650 cfs was used for design. Since water discharges of 486 cfs are expected at much higher frequencies than the design discharge (8,000 cfs), air velocities have been limited to 75 fps for the low flow system rather than 150 fps, the maximum air velocity recommended in EM-1110-1602. Justification for this reduction in maximum air velocity

should be investigated in more detail at the FDM level. Two 2-foot-diameter conduits will supply air from the main air supply conduit. Air will be distributed from the roof at the aeration offsets using a 2.33-foot by 3-foot plenum.

i. Downstream Transition. The two regulating outlet conduits, each 5 feet wide by 9 feet high, and the 2-foot-wide by 3.5-foot-high low flow bypass conduit symmetrically transition to an 18-foot-diameter horseshoe section (see plate 4-5). The transition takes place over a length of 205 feet beginning at the offset section. Four and one-half feet downstream of the RO gates, the 5-foot by 9-foot section expands to a 6-foot-wide by 18-foot-high section by expansion of the roof, a 1-foot offset on the floor, and a .5-foot offset on each wall. Two piers separate the conduits for a distance of 120 feet downstream of the offset. The tunnel width remains constant at 24 feet for a distance of 140 feet downstream of the offsets. Over the length of the piers, the RO conduits expand to 7 feet wide, the low flow bypass conduit expands to 5 feet wide, and the pier thickness decreases to 2.5 feet. A 50-foot-tangent section is provided downstream of the pier nose and then the conduit transition from a 24-foot-wide horseshoe-shaped section to an 18-foot-diameter horseshoe section over the next 65 feet. The length of the piers is controlled by the minimum discharge line. Downstream of the gates the minimum discharge line is located within one of the piers that separate the low flow bypass from the main RO conduits (see plate 4-5). The invert of the disk gate valve for the minimum discharge line is located 12 feet above the invert of the RO slide gates. Piers are designed so that flow through the minimum discharge line will make the drop in elevation to the invert of the downstream conduit without separation of the flow. Note that if the alternative for the minimum discharge line with downstream control is selected, the pier length could be shortened. A minimum pier length of 141 feet measured from the disk gate valve or 120 feet from the offsets was determined using jet trajectory equations. The rest of the transition was designed based on information provided by the Bureau of Reclamation, EM 1110-2-1602, guidance, and investigations and computations from existing projects. A hydraulic model study should be conducted at the FDM level to confirm that the transition will be hydraulically acceptable. An energy loss at the offset and through the transition of 0 percent (maximum velocity), and an energy loss of

20 percent was used for capacity design (maximum depth). These computations, from the RO gates downstream to the end of the transition, are summarized in table 6-17.

Table 6-17. Mid-Tunnel Control - 18-Foot-Horseshoe Tunnel - Velocity and Capacity Design Computations Downstream from Regulating Outlet Gates to End of Transition

Variable	Maximum Velocity	Maximum Depth
Maximum discharge (total)	8,000 cfs	8,000 cfs
Pool elevation	2,580 ft	2,580 ft
Gate opening	6.0 ft	6.3 ft
Depth at vena contracta	4.7 ft	4.9 ft
Velocity at vena contracta	172 ft/s	162 ft/s
Specific energy at vena contr.	465 ft	414 ft
N-value through transition	0.008	0.015
Percent energy loss due to trans.	0 %	20 %
Percent energy loss from friction	9 %	30 %
Specific energy at end of trans.	422 ft	207 ft
Depth at end of transition	2.0 ft	2.9 ft
Velocity at end of transition	164.5 ft/s	114.6 ft/s

j. Regulating Outlet Conduit Design. The conduit has an 18-foot-diameter horseshoe cross section. The conduit was sized based on diversion requirements. After construction the conduit will pass all regulated discharges with open channel flow. Two flow conditions were examined for the RO conduit: (1) maximum velocity to evaluate cavitation potential and for design of energy dissipator; and (2) maximum depth for design of system capacity. Depths at the upstream end of the conduit were determined on the basis of maximum and minimum energy losses through the transition. These depths were then used in the Corps H6209 program to calculate water surface profiles through the conduits. Manning's "n" values of 0.008 and 0.015 were used for velocity and capacity design, respectively. Calculations were made for a conduit length of 952 feet, portal exit at Station 27+57. For capacity computations the maximum depth in the conduit and exit chute is 6.9 feet for a

discharge of 8,000 cfs without air entrainment. Up to a 50 percent increase in volume due to air entrainment is expected. This may be conservative since air has been assumed to stay entrained in flow along the entire conduit length, even though some air may escape from flow as water velocities decrease downstream. This percentage should be used for design work until further study is performed, however. Bulking may increase maximum depth of flow (air-water mixture) to 10.4 feet in the conduit (58 percent of available conduit height). Minimum energy loss coefficients were used to calculate the maximum velocity at the portal exit of 124 fps for a discharge of 8,000 cfs. Numerical results of analysis for velocity and capacity design are listed in table 6-18. The energy grade line and pressure grade line for velocity and capacity analyses are shown on figure 6-30.

Table 6-18. Mid-Tunnel Control - 18-Foot-Horseshoe Tunnel - Velocity and Capacity Design Computations Downstream from End of Transition to Portal Exit

Variable	Maximum Velocity	Maximum Depth
Maximum discharge (both conduits operating)	8,000 cfs	8,000 cfs
Pool elevation	2,580 ft	2,580 ft
Depth at end of transition	2.0 ft	2.9 ft
Velocity at end of transition	164.5 ft/s	114.6 ft/s
N-value through conduit	0.008	0.015
Velocity at end of conduit	124.0 ft/s	64.0 ft/s
Depth at end of conduit	3.6 ft	6.9 ft
Depth at end of conduit (with 50 percent bulking)	5.4 ft	10.4 ft
Percent of conduit height (with 50 percent bulking)	30.0 %	57.8 %

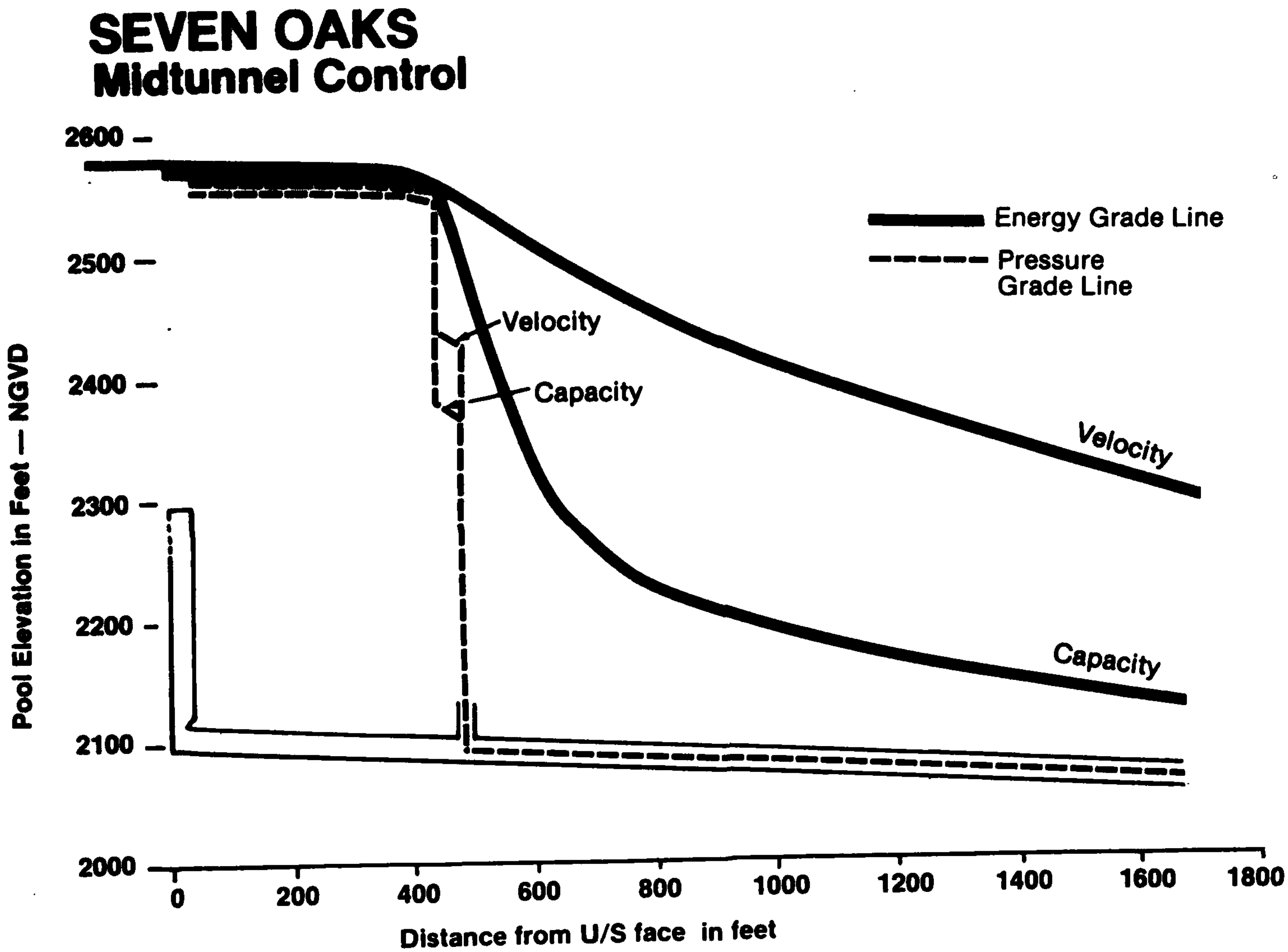


Figure 6-30 EGL and PGL for Regulating Outlet Works, Q=8000 cfs

SECTION 7

STRUCTURAL DESIGN

7.1 General. This section covers the structural design of the features for the three outlet works alternatives. The alternatives are typically comprised of an intake tower, gate chamber, diversion/outlet tunnel, access shaft/adit, control and air supply towers for mid-tunnel control, and downstream outlet and outlet channel structures. Enough preliminary analyses were performed to quantify and support a GDM level cost estimate and for feasibility of the proposed features. Information is presented which was used for the preliminary structural work and which is recommended for future design efforts. Design criteria, assumptions, conditions, procedures, and preliminary results are described in the text and in the referenced plates and figures. Brief descriptions of the three alternatives are as follows:

a. Upstream Control. The outlet works consists of a 222.5-foot-high intake tower, an 18-foot by 32-foot (inside width by height) concrete lined horseshoe tunnel, downstream outlet structure, outlet channel, and a plunge pool. The base of the tower, embedded in rock, is designed to accommodate diversion flows, wet wells, maintenance gating, and upstream RO conduit entrances. The upper 144 feet of the tower, above El. 2,156, are essentially a free standing cantilever. Flood releases pass through trash struts and over the main tower sill at El. 2,265. The sill elevation was established assuming 165 feet of sediment deposition over the 100-year project life. The top of the tower is at El. 2,302.5, and will be subjected to submergence of 275 feet for the standard project flood. Flows will then move into the 36-foot-diameter wet well to the RO entrances located at the base of the tower at El. 2,100. Two 5-foot-wide by 9-foot-high RO conduits are designed to pass the higher regulated flows. Upstream operating and emergency hydraulic slide gates are used to regulate the flow. The gates are in an independent chamber located immediately downstream of the tower base. Initially during diversion, areas within the tower base and downstream within the gate structure will be blocked out to allow for passage of diversion flows through an oversized conduit. During the second phase of construction the diversion conduit will be reduced in size to become the RO conduits. The plans and sections of the

completed intake are shown on plates 2-3 through 2-6. The base of the intake structure will be completed under the tunnel contract and will serve as an entrance intake during the diversion phase. The diversion/outlet tunnel will be excavated through rock on the left abutment and then lined with reinforced concrete. The tunnel undergoes a transition immediately downstream of the hydraulic gates, from a 25-foot-wide rectangular section to an 18-foot-wide circular bottom section. The primary tunnel support system will include steel ribs and/or rock bolts, and a thin layer of fiber-reinforced shotcrete to act as continuous blocking and support. Between the ribs shotcrete will be 2 inches minimum and 4 inches maximum. Tunnel plan, profile, and sections are shown on plate 4-2. Downstream outlet structures are located at the tunnel exit. These structures provide tunnel access, upstream gate operation, and electrical and mechanical equipment. A concrete U-channel connects the downstream outlet structures to the plunge pool. The upstream slope of the plunge pool is protected by a gravity concrete cutoff wall placed beneath the downstream end of the channel walls. The outlet channel walls, cutoff wall, and plunge pool will be completed under the tunnel contract (see construction schedule). To maximize downstream protection, the intake tower and downstream outlet structures will be completed during the dry season of the fourth year of the embankment dam construction.

b. Mid-Tunnel Control. The three alternatives have a similar intake tower. There are some differences with the bulkheading, RO entrances, and piping, yet all have essentially the same basic exterior geometry and structural design. The tunnel is an 18-foot-wide by 18-foot-high horseshoe. The tunnel will be a pressure tunnel from the upstream tower to the hydraulic gates located in the gate chamber, 430 feet downstream. The gate chamber is accessed by way of a 516-foot combination of access shaft and tower. Air for the gates is supplied from a separate 10-foot-diameter air shaft and tower. The downstream tunnel section is an open flow 18-foot horseshoe design. Construction sequence will be similar to that described for the upstream alternative. Flows will exit the tunnel and enter the outlet channel, pass over the flip bucket, and into the plunge pool.

c. Downstream Control. The outlet works consist of the similar 222.5-foot-high intake tower, an 18- by 18-foot (inside dimensioned) concrete lined horseshoe tunnel, an 11-foot-diameter steel outlet conduit, downstream outlet structure, an outlet channel with flip bucket, and a plunge pool. Within the base of the tower a single 11-foot square, transitioning to an 11-foot circular steel RO conduit, is designed to pass the higher regulated flows. The plans and sections of the completed intake are shown on plates 4-3 through 4-6. The base of the intake structure will be completed in the tunnel contract and will serve as an entrance intake during the diversion phase. Downstream operating and emergency hydraulic slide gates are used to regulate the flow. Initially during diversion, areas within the tower base and downstream within the gate structure will be blocked out to allow for passage of diversion flows through an oversized conduit. During the second phase of construction the diversion conduit will be reduced in size to become the RO conduit and summer low flows will be passed through the smaller pipes located in the tunnel floor. The tunnel will contain the 11-foot-diameter pressure conduit, transitions, and low flow system piping. Tunnel plan, profile, and sections are shown on plate 4-2. Downstream outlet structures provide tunnel access, downstream gating and control, and electrical and mechanical equipment. A concrete multi-U-channel connects the downstream outlet structures to the plunge pool. At the end of the outlet channel, a flip bucket will be provided to dissipate the regulated flows. The outlet channel walls, flip bucket, and plunge pool will be completed under the tunnel contract (see construction schedule). To maximize downstream protection, the intake tower, steel RO conduit, and downstream outlet structures will be completed during the dry season of the 4th year of the embankment dam construction.

7.2 Design Criteria and Project Conditions.

a. References. The preliminary design and future design efforts should follow accepted engineering practice and should be in accordance with the following engineering manuals (EM's), engineer technical letters (ETL's), and regulations (ER's):

EM 1110-1-2101	Working Stresses for Structural Design
EM 1110-2-2000	Standard Practice for Concrete
EM 1110-2-2102	Waterstops
EM 1110-2-2103	Details of Reinforcement - Hydraulic Structures
EM 1110-2-2400	Structural Design of Spillways and Outlet Works
EM 1110-2-2502	Retaining Walls
ER 1110-2-1806	Earthquake Design and Analysis for Corps of Engineers Projects
ETL 1110-2-256	Sliding Stability
ETL 1110-2-265	Strength Design Criteria for Reinforced Hydraulic Structures
ETL 1110-2-301	Interim Procedure for Specifying Earthquake Motions
ETL 1110-2-303	Earthquake Analysis and Design of Concrete Gravity Dams

Other applicable ETL's, EM's (EM 1110-series), and codes listed therein.

b. Basic Data. The design data is based on previously determined reservoir elevations, geotechnical exploration and interpretation, engineering judgment, and consultant input on the seismic risk for the project site. Further tests during the FDM phase of study will be required to verify material assumptions made for the GDM analyses.

(1) Water Elevations. (Operated essentially as dry reservoir.)

Maximum pool (PMF)	El. 2,604	
Standard Project Flood (SPF)	El. 2,575	
Spillway crest	El. 2,580	
100-year Flood	El. 2,535	
Normal debris pool - duration 6 months	El. 2,200	Year 0
	El. 2,300	Year 100
Maximum tailwater (8,000 cfs)	El. 2,016	

(2) Soil and Rock.

(a) Bedrock. (Dioritic, moderately hard, highly fractured.)

Unit weight	170 pcf
Deformation modulus	2.0×10^6 psi
Unconfined compressive strength	5,000 psi
Friction angle (phi)	35 - 40 degrees
Concrete/rock cohesion value (c)	100 psi
Allowable bearing capacity*	40 ksf
Subgrade modulus	1,000 kcf
Permeability	.01 - .1 ft/day

*Safety factor built in

(b) Soils.

Colluvium from left abutment:

Dry unit weight	120 pcf
Saturated unit weight	135 pcf
Allowable bearing	8 ksf
Friction angle	30 degrees
At rest lateral coefficient (Ko)	0.45

Alluvium:

Dry unit weight	133 pcf
Saturated unit weight	145 pcf
Allowable bearing	8 ksf
Friction angle (phi)	36 - 40 degrees

Processed backfill:

Moist unit weight	120 pcf
Saturated unit weight	135 pcf
Allowable bearing	8 ksf
Friction angle (phi)	40 degrees
Kr	.36

Sediment Deposits:

Dry unit weight	130 pcf
Saturated unit weight	142.5 pcf
Friction angle (phi)	32 degrees
Kr	.50

Liquefaction potential for upper 25 feet of sediment:

Buoyant unit weight	60 pcf
Kr	1.0

(3) Materials.

Concrete	f'c = 4,000 psi @ 28 days
Tunnel shotcrete	f'c = 5,000 psi
Reinforcement -	
ASTM A 615 Grade 60	fy = 48,000 psi
Steel sets - ASTM A 36	fy = 36,000 psi
Bulkheads - ASTM A 36	fy = 36,000 psi

(4) Seismic Probability. The estimated (mean) ground motion parameters and related earthquake motion data for structures at the site are as follows (reference report dated March 30, 1987, by Bruce A. Bolt, Registered Geologist and Geophysicist; Subject, Seismological Report for Seven Oaks Dam):

Estimated (Mean) Ground Motion Parameters on Rock at Site

	Maximum Credible Earthquake (Adjacent San Andreas Fault)	Maximum Probable Earthquake (in 50 years)
Source distance to dam site (km, surface rupture)	2	20
Magnitude (Ms)	8+	7.5 - 8.0
Seismic moment (dyne-cm)	10^{28}	10^{27}
Recurrence rate	150 ± 30 years	1% chance per year
Peak horizontal acceleration	0.7g	0.5g
Peak horizontal velocity (cm/sec)	90 - 105	70 - 80
Peak horizontal displacement (cm)	50 - 80	40 - 70
Bracketed duration at 0.05g (sec)	40 - 50	35
Predominant period in ground velocity (sec)	0.2 - 10.0	0.3 - 8.0

(5) General Design of Concrete Structures.

(a) Loads. All of the structures of the outlet works will be designed for the following loads and their probable combinations:

- Structure dead weight
- Uplift
- Water
- Sediment
- Backfill
- Seismic acceleration
- Wind (30 psf)
- Debris
- Floors/decks

stairs and landing	100 psf or 1 kip conc.
gratings	200 psf
hoists/rails	15 kips
access decks	truck/crane or 500 psf
gate room floor	200 psf

(b) Overturning Stability. Where applicable, overturning for the outlet tower and the structures downstream is based on EM 1110-2-2200. Overturning of the intake tower is minimized as a stability concern due to the embedment of the tower into the rock formation below El. 2,156. Overturning of the tower in the upstream direction is resisted by shear, mobilized along the excavated side slopes. The intake tower is considered free from overturning and will be designed as a cantilever above approximate El. 2,156. The outlet channel walls will be analyzed as retaining walls. The mid-tunnel control tower and air supply tower are analyzed as freestanding.

(c) Sliding Stability. Sliding stability for freestanding concrete structures is designed so that the sliding resisting force will be greater than the horizontal component of the driving force by the appropriate factor of safety specified. The factor of safety for sliding will be computed in accordance with ETL 1110-2-256, "Sliding Stability for Concrete Structures," as follows:

$$S.F. = \frac{V \tan \phi + cA}{H}$$

where:

V = sum of all vertical forces

ϕ = angle of internal friction

c = cohesion of interface

A = area of the horizontal plane considered

H = sum of all horizontal forces

The minimum required factor of safety against sliding for normal static loading conditions is 2.0. The minimum required factor of safety for seismic loading conditions is 1.3. For extremely unusual combinations, a minimum factor of safety of 1.0 may be applied.

7.3 Intake Tower.

a. General. The dimensions and shapes of the structure were developed in the following manner (see plates 2-3, 3-3, and 4-3):

(1) The lower tower section is embedded into a sound dioritic rock formation. The rock surface rapidly dips upstream and toward the valley. This factor established the limit of the intake rock excavation and tunnel cover. Intake excavation is minimized by using near vertical slopes below El. 2,156 and 1H on 4V cut slopes with no benching elsewhere.

(2) The base is determined by the requirements to satisfy the hydraulic functions of diversion, RO, and low-flow release; arrangement of maintenance gating; and design thicknesses for stability and to resist the expected loads.

(3) The tower itself is designed to pass the regulated outlet flowswhile satisfying hydraulic criteria, provide bulkheading capability, and resist applied loads, particularly the combinations of seismic and sediment.

(4) The intake trash structure must have enough openings to pass flow at a maximum velocity of 10 fps. Opening size is limited to two-thirds the least RO conduit dimension. Additional openings were provided as a safety factor against debris plugging. The trash struts are designed to withstand a plugging pressure differential of 20 feet.

b. Load Cases. Applicable load conditions for intake tower design are as follows: the case numbers are based on the loading conditions outlined in EM 1110-2-2200. In general, under normal operating conditions the tower essentially has no overturning moments and its height does not present any excessive bearing stress problems.

(1) CASE I - Construction Condition.

(a) Reservoir empty
(b) Structure dead load
(c) Wind
(d) Sediment between Els. 2,100 and 2,265

(2) CASE II - Normal Operating Condition.

(a) Debris pool Els. 2,200 to 2,300
(b) Structure dead load
(c) Wind
(d) Sediment between Els. 2,100 and 2,265
(e) Uplift

(3) CASE III - Induced Surcharge Condition.

(a) Standard project flood (SPF) El. 2,575
(b) Structure dead load
(c) Wind
(d) Uplift
(e) Sediment at Els. 2,100 to 2,265

(4) CASE IV - Flood Discharge Condition.

(a) Probable maximum flood (PMF) El. 2,604

(b) Structure dead load

(c) Wind

(d) Uplift

(e) Sediment at Els. 2,100 to 2,265

(5) CASE V - Construction Condition with Earthquake.

(a) Reservoir empty

(b) Structure dead load

(c) Earthquake (X or Y axis, bidirectional)

(6) CASE VI - Normal Operating Condition with Earthquake.

(a) (aa) Debris pool Els. 2,200 to 2,300

(ab) Structure dead load

(ac) Earthquake (X or Y axis, bidirectional)

(ad) Sediment at Els. 2,100 to 2,265

(b) (ba) 100-year high pool El. 2,535

(bb) Structure dead load

(bc) Earthquake (X or Y axis, bidirectional)

(bd) Sediment at Els. 2,100 to 2,265

c. Earthquake Design and Analysis.

(1) General. ER-1110-2-1806 requires seismic design to be considered for new structures that retain or have the potential to retain a permanent pool. If a reservoir could be lost due to failure caused by earthquake loading, and the result would cause property damage and/or loss of life, seismic design is required. ETL-1110-2-303 states "...the pool level selected for an earthquake loading case should normally be a pool level which occurs, on the average, relatively frequently during the course of the year." For the Seven Oaks Intake Tower, a variety of debris

pool and sediment levels will be analyzed as normal operating conditions. A high pool (10-year - El. 2,400) coupled with an operating basis earthquake is an example of a stability condition considered as an extremely unusual event.

(2) Stability Analysis Requirements.

(a) Static Analysis. Where applicable, simplified static force approximations and methods of analyses are used for the stability computations. EM 1110-2-2200 requires a stability analysis by a seismic coefficient method. The seismic coefficient is generally found in ER 1110-2-1806. The intake tower is located in Zone 4 as shown on the seismic zone map of the contiguous States and Puerto Rico. The corresponding seismic coefficient would be 0.20. Due to the close proximity to the San Andreas Fault and the site specific data as developed, larger seismic coefficients as high as one-half of the peak ground acceleration will be considered in combination with safety factors of unity for extremely unusual load conditions. Stability of the tower is analyzed for movement in the upstream direction only, as other directions are resisted by the excavated rock face. For the upstream earthquake scenario, the tower, by itself ("freestanding"), will not satisfy stability criteria. Base shear (OBE) will vary from 26,000 kips to well over 100,000 kips depending on pool elevation, the static coefficient used, and the method of predicting earthquake forces. Shear, as estimated by the Westergaard formula increases dramatically with reservoir depth. The resulting high overturning moment will cause the resultant to fall outside of the base. Where tension zones are created, cohesion between the rock and concrete will be neglected. Without the tower base in compression the resistance to sliding isn't adequate to achieve acceptable sliding safety factors. To reduce the shear and moment at the tower base, the support provided by the side slopes and foundation excavation must be utilized. To utilize the side slope resistance, several construction actions may be required. There are several methods to ensure mobilization of side slope resistance. Some of which are: (1) grouting the concrete/rock interface to assure cohesion; (2) provide rock anchorage to mobilize shear friction; (3) provide a vertical concrete key; (4)

post-tensioning to mobilize friction; and (5) do nothing (existing cohesion and surface disparities will provide adequate resistance). Further modeling and analysis will be required during the feature design efforts to qualify the solution best suited for this structure. The load conditions and static stability criteria are as follows:

Table 7-1. Earthquake Load Conditions and Static Stability Criteria for Intake Tower Movement in the U/S Direction

Case	Load Condition	Sediment Elevation	Static Coef.	Base Area In Comp	SSF
(V)	Dry + OBE	2,100	.25	resultant	1.3
(V)	Dry + MCE	2,100	.35	within	1.0
(VI)	2,200 dp + OBE	2,100	.25	base	1.3
(VI)	2,200 dp + MCE	2,100	.30	"	1.0
(VI)	2,300 dp + OBE	2,100 - 2,265	.20	"	1.0

NOTE: OBE - operating basis earthquake
MCE - maximum credible earthquake
dp - debris pool

For higher debris pool scenarios, the tower will be embedded in sediments which appear to stabilize the structure under the static type of analysis. A finite element analysis is recommended to determine the tower stresses under the high debris pool conditions. These studies will assist in predicting the tower-sediment-reservoir interaction and the resulting tower stresses.

(b) Static Earthquake Forces. The lateral inertia force on the tower will be a percentage of the structure weight applied at the center of gravity of the tower. The percentage will correspond to the static coefficient assigned to the load condition under study. The added and subtracted seismic water forces will be based on the Westergaard

parabola for submerged and partially submerged structures. A sensitivity study should be performed on the different methods for predicting earthquake loading on the tower. For a tower with a varying cross section, an added mass two mode analysis may give a more reasonable estimate of lateral forces than Westergaard's formuli. Finite element and other structural modeling can be used to further quantify the expected loading. The added soil or sediment force on the tower and/or other structures is KhW, acting at the center of gravity of the soil wedge in accordance with the current draft EM, "Retaining and Flood Walls." Kh is the static coefficient of horizontal earthquake acceleration chosen for the load condition being studied. W is the weight of the material in the wedge, including water. Liquefaction of the upper 25-foot sediment zone is also to be considered during a seismic event.

(3) Stress Analysis.

(a) Requirements. ER 1110-2-1806 requires a dynamic response type of stress analysis for concrete structures in seismic zones 3 and 4. The Seven Oaks site and tower satisfies all conditions requiring a dynamic analysis. Where stresses/forces/moments must be determined by dynamic analysis, current Corps guidance directs preliminary calculations be made using a simplified response spectrum method of analysis. For the preliminary GDM analysis, a Two-Mode Added Mass Analysis was performed (Chopra, 1981). The procedure was followed as outlined in the WES report by Mlaker and Jones, dated 1982; Subject, "Seismic Analysis of Intake Towers." The method computes the maximum earthquake loading for the linear response of the tower in its first two fundamental modes of vibration to the horizontal component of the ground motion.

(b) Dynamic Analysis. The hydrodynamic effects are modeled as an added mass of water moving with the tower. The method of added masses does not predict a significant increase of hydrodynamic forces on the tower due to submergence. The added mass of the tower is primarily a function of the tower section and does not change appreciably with submergence. There is some variation of the added mass ratio near the tower top, but at the bottom of the tower it remains unchanged.

Investigation of Westergaard's formulas for hydrodynamic earthquake pressures show a marked increase of the parabolic water mass moving with the tower under submerged conditions. Specifically, in that the hydrodynamic force is directly a function of the water submergence over the tower; $F = 36.5H^{1/2}h^{3/2}a/g$ (H = water depth, h = tower height). To consider the effects of submergence possibily not considered within the added mass analysis, the added masses were increased by 50 percent for the 100-year flood condition. The question of hydrodynamic forces under submergence will be addressed further under the Feature Design Memorandum phase of study. The earthquake loading is computed directly from the spectral acceleration, obtained from the response spectrum, and dynamic properties of the structural system. Seed's response spectrum (mean) for 28 rock records and 5 percent damping was scaled for the earthquake under analysis. The resulting load is applied as an equivalent static loading and a conjugate beam procedure is employed to determine the shears and moments. A program in BASIC was written to ease the numerical computations of the added mass analysis. The program computes the first and second frequencies of a non-uniform cantilever using Rayleigh's method. The tower is divided into sections with corresponding section masses, added masses, and stiffnesses. Once convergence has been achieved for the tower frequencies, pseudoabsolute accelerations (scaled for desired ground motion) are input into the program. Inertia forces, shears, and moments are computed and applied in a conjugate beam analysis. The final equivalent modal forces are combined as probable maximums using a root-mean-square approximation. Typical reinforcement for a debris pool plus OBE load condition (for the circular tower section) is shown on plate 2-3 and moments and shears on figure 7-1.

(c) Tower Limitations. Reinforcement and section requirements for the circular section under seismic loading will control the feasibility and cost of a tower at this site. A higher tower than proposed or a similar tower under higher seismic loading than assumed may well be infeasible for this site. As moments and shear increase, a larger tower section is required to carry the increased loading. The larger tower section results in a further increase of inertia and hydrodynamic forces. Iteration to achieve a satisfactory design may lead to a

non-converging solution. The sections of the proposed tower were determined through this iterative process under the given assumed loading conditions. To accommodate higher loading scenarios, other tower alternatives are available: an inclined tower, increasing capacity by post-tensioning the existing design, removing the sediment deposition allowing a shorter tower; or utilizing tower staging as the sediment level rises.

(d) Load Conditions. Loading conditions requiring a stress analysis (reinforcement design) are listed in table 7-2. The table of load conditions and design criteria represent "preliminary GDM design thinking" and are not meant to be considered as final or all-inclusive. Where extremely unusual load conditions are considered the ultimate strength design, load factors are reduced (allowable stresses increased).

Table 7-2. Reinforcement Design Load Conditions

Load Condition	Sediment Elevation	Concrete Design USF LF
Debris pools		1.5DL+1.9
no earthquakes	2,100 to 2,265	(other forces)
High pools (hp)		
no earthquakes	2,100 to 2,265	reduced
Dry + OBE	2,100	.75 (1.9)E
Dry + MCE	2,100	.75 (1.7)E
2,200 dp + OBE	2,100	.75 (1.9)E
2,200 dp + MCE	2,100	.75 (1.33)E
2,300 dp + OBE	2,265	.75 (1.7)E
2,300 dp + MCE	2,265	.75 (1.33)E
100-year hp + OBE	2,100 to 2,265	.75 (1.5)E

The design of miscellaneous structural and metal features and electrical and mechanical equipment must also consider the forces induced by seismic vibrations. Typical features such as stairway systems, equipment platforms, base connection for the hydraulic gates, hydraulic piping, switching, conduits, and bridge bearing details must be designed for earthquake induced stresses.

INTAKE TOWER
TWO MODE ADDED MASS DYNAMIC ANALYSIS
OBE = .5g - DP EL. 2,200

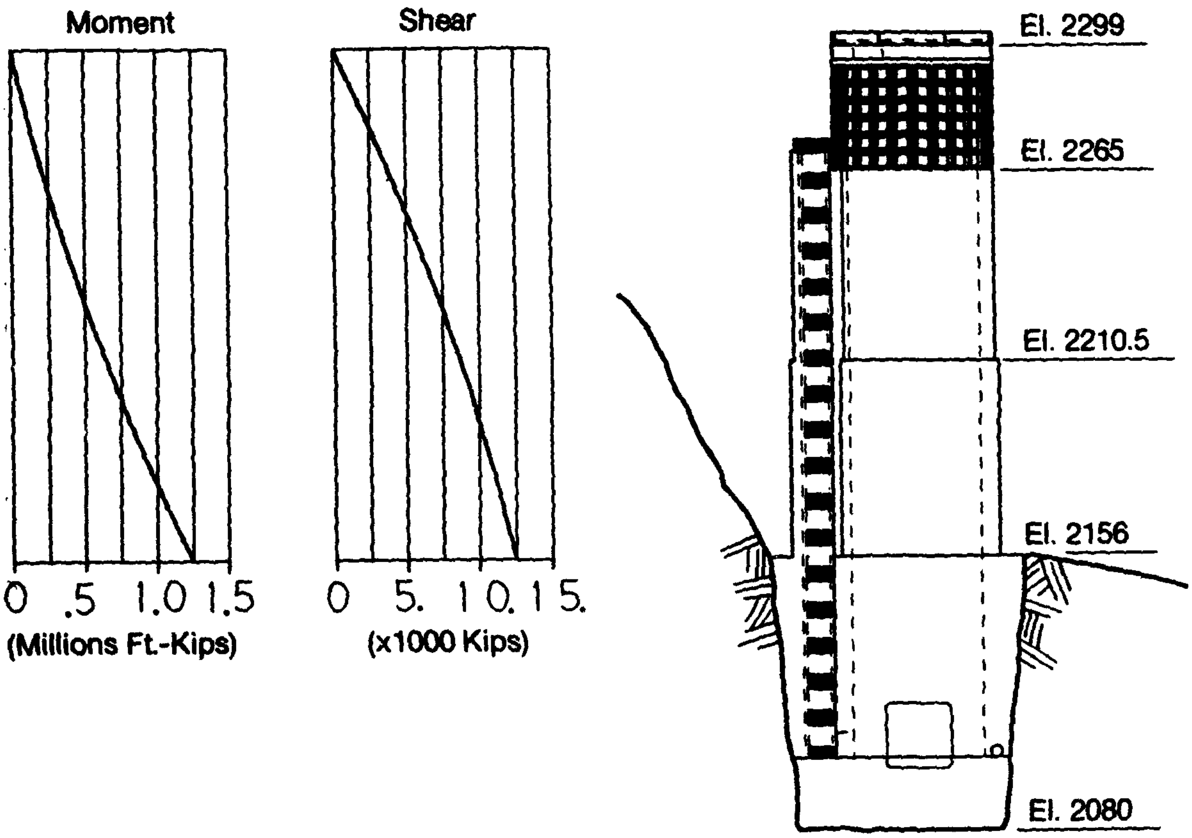


Figure 7-1

d. Access Bridge. The bridge is similar for all three control alternatives and is a single-lane, 54-foot single-span structure to be designed for normal operating HS-20 loads, a 15-ton project crane, and dump truck. Design will be in accordance with American Association of State Highway and Transportation Officials standard specifications for highway bridges. For estimating purposes, the bridge was assumed to be made up of precast "I" girders with a cast-in-place deck. Cost was based on concrete volume and square footage of bridge deck surface.

e. Regulating Outlet Maintenance Bulkhead. The RO maintenance gates for all three alternatives will be of similar construction and designed for the same operating criteria and load conditions. The upstream control gates are approximately 6 feet 10 inches by 9 feet 6 inches (two required), the mid-tunnel gate is 25 feet by 20 feet 6 inches, and the downstream gate will be roughly 14 feet by 12 feet. The gates will be of welded construction with the skin plate and seals on the downstream side (see plates 2-3, 3-3, and 4-3). It will be a slide gate designed to withstand head resulting from a pool elevation of 2,299. The gate (s) will close under their own weight under *static conditions*. The gate will be stored in the slot. End girders will be sufficiently flexible to assure direct bearing of load-carrying members without the condition of extremely close tolerances of bearing surfaces, but sufficiently stiff to assure rigidity of the bulkheads. The RO bulkhead will be constructed of ASTM A-36 steel. An RO conduit fill pipe will be required to equalize the head on the bulkhead prior to removal.

7.4 Diversion and Outlet Tunnel.

a. General. The tunnel is approximately 1,625 linear feet in length for all alternatives. It is mined by drill and blast methods through a mostly diorite formation in the left abutment (see plate 2-2). The concrete lined section is established by diversion requirements. The overall excavation is controlled by the needs of tunnel support. The tunnel support system must consider several different loads (construction and permanent) and their possible combinations. The excavation at the

floor of the tunnel is flat, creating a horseshoe tunnel. The final tunnel layout is divided into three zones which are controlled by the loading condition for each zone. The primary tunnel support during construction will be steel sets with fiber reinforced shotcrete as continuous blocking and lagging. Untensioned rock bolts will be used in the crown of the tunnel on an "as needed basis." Excavation will proceed with a single heading and full face drilling and excavation. For the upstream control tunnel (plate 2-2), construction will likely require two benches as the tunnel height is excessive at 40 feet for a single bench. Rib sets will require a wall plate with tiebacks to temporarily support the sets for the upper bench. The mid-tunnel control alternative will require a two-heading operating to meet schedule demands and minimize construction interference (see construction schedule, section 5).

b. Design Loads.

(1) Load Conditions. The following load conditions will be used for the tunnel:

(a) structure dead load
rock load

(b) structure dead load
normal external hydrostatic

(c) structure dead load
rock load
normal external hydrostatic

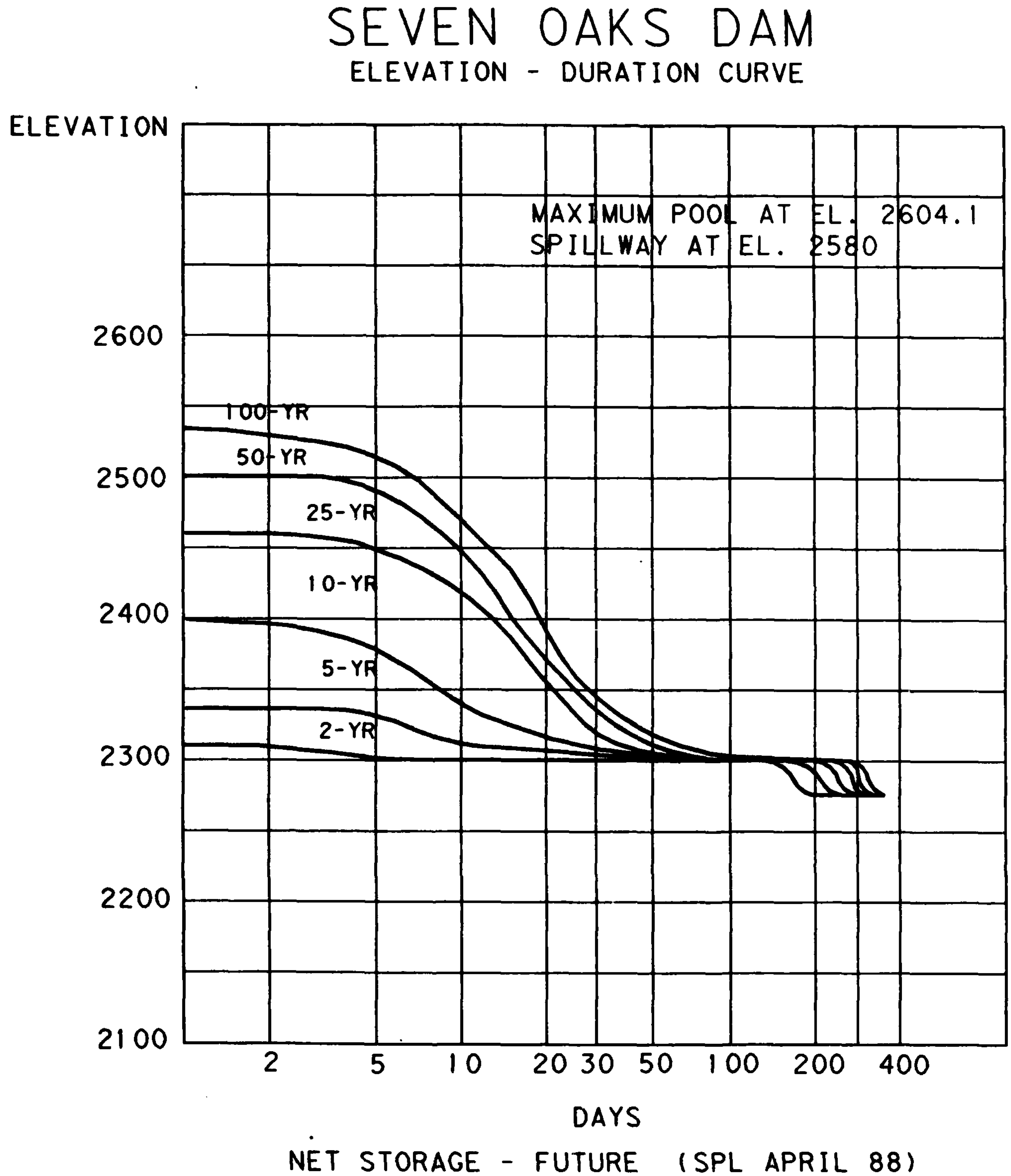
(d) structure dead load
rock load
extreme or unusual external hydrostatic loads

(e) structure dead load
rock load
seismic loads

(f) structure dead load
rock load
normal external hydrostatic
seismic loads

(2) Steel Rib Sets. Rock loads for the steel rib sets are based on a combination of the Terzaghi Classification System as developed at the University of Illinois by D. U. Deere. The rock conditions for the tunnel have been assumed to be a class no. 4 or 5, "very blocky or seamy." For this level of design, steel sets have been assumed for the entire tunnel length. The majority of the tunnel proper is designed assuming the full vertical rock load of .5(B+H)Wr, where B is the width of the tunnel, H is the height, and WR is the rock density. This load assumption is used for the rib set design as well as the concrete liner. The tunnel portals will be designed for the larger load of 2.5 B^2Wr. These assumptions will be subject to change as further design and explorations are accomplished. Assumed tunnel rock loads will account for abnormal residual rock stresses if found to be present.

(3) Concrete Tunnel Lining. The concrete lining has been designed for an unbalanced vertical rock load equal to one-half the load used for designing steel set supports and for a further reduction when the rock bolt/shotcrete support system is used. The rock load assumptions are preliminary; further extensive study of the rock at the site, exploration laboratory testing, and tunneling method and construction evaluation will be required. In addition to the rock loads mentioned above, the design must include hydrostatic load due to the expected pool elevations (see figures 7-2 and 7-3), even though rock permeability is expected to be low. As the figures show, the tunnel has been divided into three design zones with respect to hydrostatic conditions and physical location and layout. Where rock drains (tunnels) are used, the maximum reduction in hydrostatic loading is assumed as 50 percent. The drain effectiveness may increase (75 percent) when drain reliability and rock conditions are further qualified during FDM design efforts. The rock is highly fractured but is typically very "tight." Initial permeability of the rock mass surrounding the tunnel has been assumed (based on boreholes in similar



SEVEN OAKS DAM
ELEVATION - DURATION CURVE
ELEVATION
MAXIMUM POOL AT EL. 2604.1
SPILLWAY AT EL. 2580
2600
2500
2400
2300
2200
2100
100-YR
50-YR
25-YR
10-YR
5-YR
2-YR
2
5
10
20
30
50
100
200
400
DAYS
NET STORAGE - FUTURE (SPL APRIL 88)

FIGURE 7-2

SEVEN OAKS - TUNNEL DESIGN
D/S Control - Concrete Liner
Hydrostatic Loading
GDM Study Level

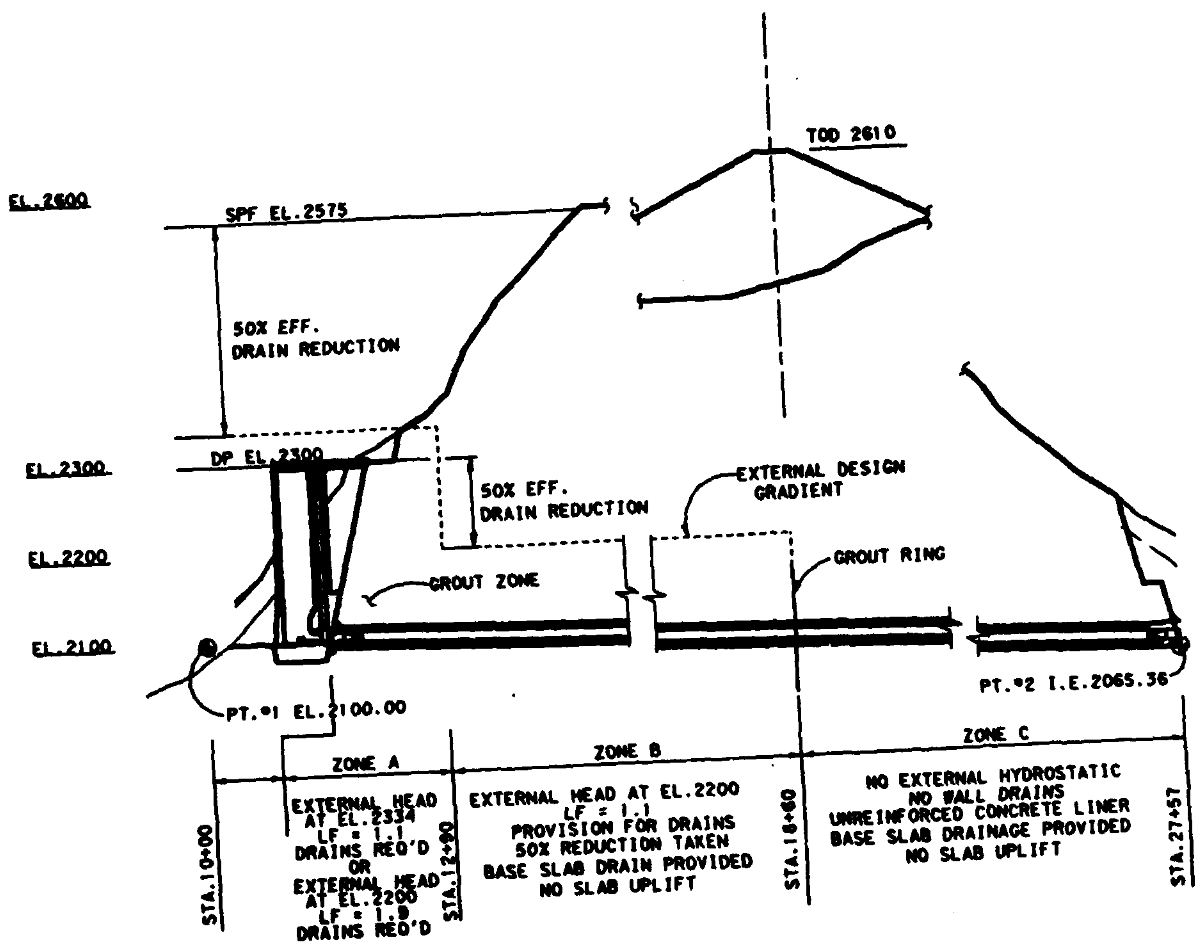


Figure 7-3

rock) at approximately 0.03 feet per day and can vary from 0.1 to 0.01 feet per day. Considering the frequency of high pools combined with the rock tightness, the following external load cases were picked for the three zones:

(a) Zone A - Upstream transition

Upstream control - Station 10+85 to Station 13+15
Mid-tunnel - Station 11+30 to Station 13+10
Downstream - Station 11+40 to Station 12+90

- normal "year zero" debris pool, El. 2,200
- 50 percent reduction drains required
- normal design load factors LF = 1.9

or

- standard project flood, pool at El. 2,575
- 50 percent drain reduction
- allowed stress increase, LF = 1.1

(b) Mid-zone B

Upstream control - Station 13+15 to Station 19+00
Mid-tunnel - Station 13+10 to Station 18+60
Downstream - Station 12+90 to Station 19+00

- normal debris pool, El. 2,200
- reduction for drains
- provisions for drains
- hydrostatic pressure monitoring

(c) Zone C

Upstream control - Station 13+15 to Station 27+50
Mid-tunnel - Station 18+60 to Station 27+57
Downstream - Station 19+00 to Station 27+57

- no external hydrostatic
- no drains
- unreinforced concrete liner

Consolidation grouting will be performed at the tunnel entrance portal to reduce potential seepage through the rock disturbed by the excavation process. A grout ring will be placed at Station 19+00 to control seepage past the dam centerline. Monitoring of liner back pressure will be required for zones B & C.

c. Design Methods.

(1) Steel Rib Sets. The steel rib sets have been designed by the procedures (Proctor and White) given in EM 1110-2-2901. Rock loads are transferred through blocking to the rib sets. The shotcrete is assumed to provide a near continuous blocking. A uniformly loaded rib was assumed for the rib set design, although this will be adjusted should abnormal residual stresses be found in the bedrock. For the upstream control alternative, at the Contractor's option, a circular or rectangular tunnel base may be mined. A ring beam or rib and post installation will be used.

(2) Concrete Tunnel Lining. The concrete lining has been modeled with the WESLIB computer program CFRAME which allows the use of elastic supports. The elastic supports model the passive resistance of the surrounding rock mass. Various subgrade modulus of the rock are used as part of the parameter sensitivity analysis. Varying hydrostatic loads were investigated to model the effect of drain location within the section. The preliminary results on which the GDM estimate are based require liners as shown on plates 2-2, 3-2, and 4-2. The liner thickness increased with the tunnel height. External hydrostatic pressures created high bending stresses in the side walls. Base slabs are typically thicker. Excavation and shotcrete quantities were based on 2 inches of shotcrete behind the rib sets and thicker shotcrete built-up around the set to provide continuous blocking.

(3) Further Analyses Required. Through feature and plans and specifications phases of design, additional analyses and procedures will be performed. Those analyses may include but are not limited to:

(a) STRUDL - elastic analysis providing for larger stiffness and loading matrix (versus CFRAME).

(b) NEWTUN - program for design and analysis of cast-in-place tunnel linings.

(c) FEM - finite element analysis, potential for two- and three-dimensional analysis - input of seismic time histories, wave forces propagated within rock, study of section capacity to resist postulated seismic 4-foot displacement.

d. Tunnel Design - General. The tunnel is generally divided into three design zones, A, B, and C. The tunnel in zone A receives the maximum rock and hydrostatic loading. Zones B and C have reduced loading assumptions due to their location relative to the reservoir, rock mass, and embankment dam core (see figure 7-3). The concrete liner is divided into 30-foot monoliths in zone A, and 40-foot monoliths in zones B and C. The contraction joints will have waterstops. Steel ribs will be the primary rock support for all zones. Encountered shear zones may require overexcavation for specific shear zone treatment. Benefits of shear zone treatments will be investigated further during the Feature Design Memorandum design efforts. Details of tunnel design for the different alternatives are presented as follows:

(1) Upstream Control. The tunnel in zone A houses the primary upstream RO conduit transitions. As such, this portion has the largest section at 35 feet wide and 40 feet high, tapering to 29 feet wide. Zones B and C maintain a constant horseshoe section with excavation dimensions of 23 feet by 35 feet and 22 feet by 34 feet, respectively. A typical reinforced section and tunneling sequence are depicted on plate 2-2.

(2) Mid-Tunnel Control. The upstream zone A is a pressure horseshoe tunnel with the typical 18-foot-wide by 18-foot-high inside dimensions. No transitions are required until the approach to the gate chamber. Lining thickness is controlled by the combined external loading of the rock and hydrostatic pressure. The stresses of side walls and floor of the tunnel are sensitive to the bending stresses created by this loading. Zone B houses the mid-tunnel gate chamber and associated transitions. This zone has reduced external hydrostatic pressures, but

significantly increased rock loads due to the increased tunnel widths through the transitions and main chamber. Excavation of the gate chamber will require a specific sequence as portrayed in figure 8-1. Rib sets will be required for the initial wider tunnel section through this reach. For the remaining excavation of the gate chamber, shotcrete, and patterned rock bolting will provide the support until the final concrete liner is placed. Zone C is a lightly reinforced open channel horseshoe with the typical 18-foot inside dimensioning.

(3) Downstream Control. A smooth concrete tunnel liner for the downstream control option will be required as a hydraulic conduit only for the period during construction diversion. After this period the liner is no longer required for hydraulic purposes, the tunnel then acts as an access adit for inspection of the RO pressure conduit. The cofferdam provides 18-year protection which can result in temporary pressurized tunnel conditions with 45 fps velocities and 60+ feet of internal pressure. The tunnel in zone A contains the main RO conduit transition and upstream concrete plug. Zones B and C maintain a constant horseshoe section with excavation dimensions of approximately 20 feet 4 inches by 23 feet 8 inches. A typical reinforced section and tunneling sequence are shown on plate 4-2.

7.5 Regulating Outlet Conduits - Downstream Control Alternative.

a. Eleven-Foot-Diameter Conduit. The main RO conduit will be designed for an internal pressure equal to the static pressure due to maximum reservoir pool plus water hammer. Water hammer pressure will be equal to a percentage of static pressure at the downstream gates decreasing linearly to zero at the intake. The conduit will be designed to span between W-section columns which bear on a Teflon surface. The Teflon is provided to allow a marginal transverse movement of the tunnel without disrupting operations. A concrete slab is required under the bearing pads to assure distribution of stresses into the foundation. One thousand psi is typically used as an allowable bearing stress for the concrete and 10 ksf is assumed as the allowable for the unconfined gravel drain material. An alternative support under consideration is a

continuous concrete saddle to embed the lower one-fifth of the conduit, as shown on plate 3-2. The conduit will be modeled and analyzed for system vibration and earthquake induced stresses and deformation during the FDM phase. A 1/16-inch of the conduit is provided as a wearing surface (sediment abrasion). Actual use of the main RO will be limited. The smaller pipes will accommodate project releases for the majority of the time. Conduit steel will conform to ASTM designation A-516, grade 60, firebox quality, with an allowable unit stress at 15,000 psi. Hydrostatic tests of all conduit sections will be made at 150 percent of the design pressure. Girth welds not subject to hydrostatic tests will be radiographed. Stiffener rings will conform to applicable provisions of ASME code for unfired pressure vessels. Expansion details will be required for longitudinal deformation due to thermal changes.

b. Low Flow Bypass and Minimum Discharge Lines. Preliminary design assumes that A-36 steel will be used for the pipes with a 1/2-inch wall thickness. The exterior will be painted and wrapped for corrosion protection. The pipe has an additional 1/8-inch wall thickness for wear due to sediment passage. Sediment passage at this project is expected to be minimal due to the intake design and reservoir trapping effectiveness. Both pipes will be designed for full pool loading, including water hammer.

7.6 Downstream Outlet Structures. The downstream outlet structures will be reinforced concrete monoliths founded upon sound rock. They will be designed for loadings as listed in paragraph 4.2.b(5)(a), "Loads." Functions vary for each control alternative. Upstream control downstream outlet structures provide tunnel access, operation and electrical equipment, mechanical equipment, power and emergency power. Downstream control downstream outlet structures provide the same functions as for upstream control with the addition of gating. For mid-tunnel control the above functions are provided by the access shaft and tower.

7.7 Outlet Channel and Cutoff Walls.

a. General. The outlet channel walls will be cantilever U-shaped walls placed both on rock and backfill foundations (see plates 2-6, 3-6, and 4-7). The walls for the downstream control alternative will be constructed in two phases; the tunnel contractor will construct the outer small 3-foot-wide channels for later small flow diversion. The channel formed between these structures will be used for diversion between construction phases (see plate 3-6). The final wall configuration will be constructed during RO conduit installation while summer flows are diverted through the low flow bypass lines. The cutoff wall will be essentially a gravity wall placed at the upstream end of the plunge pool (plate 4-7).

b. Loads. Loads will be structure dead load, backfill, hydrostatic, velocity uplift, and seismic (using seismic coefficients). Loading conditions and design will be in accordance with EM 1110-2-2502 and the draft EM, "Retaining and Flood Walls."

SECTION 8

GEOTECHNICAL DESIGN

8.1 Physiographic Setting. Rapid uplift and downcutting of the San Bernardino Mountains has produced steep-sided, narrow valleys with deep alluvial fill as exemplified by the Santa Ana River Canyon. At the dam site, the river has dissected an east-northeast trending diorite rockmass to approximate El. 1,960, NGVD. The valley subsequently has been aggraded to the present valley floor at approximate El. 2,060. The left abutment, through which the outlet works tunnel will be excavated, is a "ridge and swale" topographic feature with steep, rugged slopes averaging 1V on 1.5H. The upstream diversion channel and the downstream plunge pool will be excavated in the valley floor fill.

8.2 Exploration.

a. General. For this design stage, explorations for the outlet works were confined to the inlet and outlet portal areas and energy dissipator site. The primary purpose of the investigations was to determine the overburden thicknesses. To this end, a combination of exploratory drill holes and refractive seismic surveys were used.

b. Exploratory Drilling. Exploratory drilling for the outlet works consisted of two core holes and two rotary holes at the inlet portal area, one inclined and three vertical core holes, one bucket auger hole at the outlet portal, and one rotary and one core hole in the energy dissipator. All of the rotary-wash holes were originally planned as core holes, however, the depth to bedrock in holes R86-06 and R86-31 was deeper than anticipated and hole R86-08, in the energy dissipator area, was abandoned at 68.5 feet when the roller bit broke off in the hole. The hole locations are plotted on plate 8-1. The logs of drill holes are shown on plates 8-2 and 8-3. Pertinent hole data is tabulated below. Hole C86-24 at the outlet portal was an angle hole inclined 60 degrees on a bearing of N50E.

Outlet Works - Inlet Portal

Hole No.	Date Complete	Hole Elevation	Total Depth	Feet Cored
R86-06	10/22/86	2,136.7	72.8	49.3
C86-07	10/25/86	2,136.6	90.8	43.9
C86-30	04/16/87	2,121.9	120.4	0
R86-31	04/18/87	2,137.5	101.0	0
	Total Footages:		385.0	93.2

Outlet Works - Outlet Portal

Hole No.	Date Complete	Hole Elevation	Total Depth	Feet Cored
C86-12	11/13/86	2,135.0	102.0	27.7
C86-13	11/15/86	2,135.0	76.1	15.5
C86-24	01/22/87	2,135.0	119.4	85.9
C86-25	01/24/86	2,135.0	48.4	6.8
LAH-05	12/01/86	2,130.0	19.0	0
	Total Footages:		364.9	135.9

One bucket auger hole, LAH-5, was drilled nearby to the rotary and core holes at the outlet portal. The hole was not advanced beyond a depth of 19 feet because of severe caving in the dry colluvial overburden and bedrock was not encountered.

Outlet Works - Energy Dissipator

Hole No.	Date Complete	Hole Elevation	Total Depth	Feet Cored
R86-08	10/28/86	2,021.0	68.5	0
C86-10	11/04/86	2,021.0	126.8	6.8
	Total Footages:		195.3	6.8

Comparisons of rock quality parameters derived from this exploratory drilling data are presented in paragraph 8.6.b.

c. Geophysical Surveys. Two refractive seismic surveys, 87-03 and 87-05, were conducted in the area of the inlet portal in an attempt to determine the configuration of the bedrock under the proposed intake structure. Similarly, two surveys, 86-03 and 86-08, were conducted on the colluvium covered slopes in the area of the outlet portal in order to determine the depth to bedrock. One long refractive survey, 84-1/2, was conducted for the same reason in the area of the proposed energy dissipator. See plate 8-1 for the location of the geophysical survey lines, and plate 8-5 for the time-distance graphs and interpretations. Downhole seismic surveys were not conducted in holes associated with the outlet works.

(1) Inlet Portal. At least four velocity zones were identified on refractive seismic lines conducted at the inlet portal area. The velocity range for layer 1 (V1) represents dry, unconsolidated alluvium; layer 2 (V2) may represent more dense, unsaturated alluvium; layer 3 (V3) probably represents saturated alluvium; and layer 4 (V4) represents bedrock. Correlation of refractive seismic data and drilling data from hole C86-30 is not good and the subsurface velocity model may be more complex. The measured velocities (V) and the generalized depths (D) in feet are presented below.

Outlet Works - Inlet Portal

<u>Results of Refractive Seismic Surveys</u>

Line No.	V1	D1	V2	D2	V3	D3	V4
87-03	500 - 1,800	10	4,000	40	6,650	75	12,000
87-05	650 - 1,800	15	4,400	35	7,650		

(2) <u>Outlet Portal</u>. The refractive seismic lines at the outlet portal were both conducted on the colluvium covered slopes in order to roughly determine the overburden thickness. The velocity distinction between the unsaturated colluvium (V1) and the bedrock (V2) is quite distinct and the depth to bedrock (D1) correlates well with drill hole data. The measured velocities (V) in feet per second and the depth to bedrock (D1) in feet are presented below.

Outlet Works - Outlet Portal

<u>Results of Refractive Seismic Surveys</u>

Line No.	V1	D1	V2
86-03	675 - 2,000	65	12,000
86-08	700 - 2,100	55	9,500

(3) <u>Energy Dissipator</u>. The refractive seismic line conducted in the energy dissipator area yielded a velocity profile very similar to other lines conducted in the stream bottom. Three separate velocity zones corresponding to unsaturated alluvium (V1), saturated alluvium (V2), and bedrock (V3) are identifiable. The measured velocities in feet per second and generalized interface depths are presented below.

Line No.	V1	D1	V2	D2	V3
84-1/2	2,000 - 3,000	20	8,000	100	12,000

8.3 Laboratory Testing. The results of laboratory tests conducted on drill core samples from the outlet works are presented in paragraph 8.6.c. Petrographic analyses were not conducted on any samples from the outlet works. Petrographic descriptions in this report were adapted from analyses of similar rock elsewhere on the project.

8.4 Bedrock Configuration.

a. Upstream Portal Area and Diversion Channel. Rock is exposed in the upstream portal area on an average slope of 1V on 1H. The rock surface continues beneath the valley fill on the same slope to about El. 2,060 near hole R86-06, where it flattens to about 1V on 2H. This change in slope is probably of local extent and will not impact excavation of the diversion approach channel since it lies below excavation grade at this location.

b. Downstream Portal Area and Plunge Pool. Bedrock is covered with colluvium in the downstream portal area and valley-fill alluvium in the plunge pool area. The colluvium lies on a 1V on 1H rock slope. It varies in thickness from 0 feet at about El. 2,315 to 60 feet near the toe of the slope, where it blends into the valley fill. Seismic refraction data indicates that about 120 feet of valley-fill alluvium overlies bedrock in the plunge pool area. Rock at this location is estimated to be at El. 1,900.

8.5 Foundation Conditions.

a. Overburden Properties. The valley-fill alluvium consists of streambed alluvium and older alluvium. Streambed alluvium is composed primarily of unconsolidated gravelly sands and sandy gravels with varying amounts of cobbles and boulders that have been randomly deposited in the

river channel by aggradational processes during past flooding episodes. The older alluvium is a mixture of streambed alluvium and hillside colluvium that continuously forms along the base of the slopes. This material is a poorly consolidated, crudely stratified, poorly sorted sand and gravel with occasional cobbles and boulders in a silty matrix. Colluvium collects in topographic swales or other depressions between ridges and is a mixture of slope wash and talus (removed residual soil and rock).

b. Bedrock Properties. The diorite along the outlet works is a speckled grayish-white and black, medium-grained igneous rock that originally crystallized from a deep-seated magma body that intruded older gneissic rock. In the dam site area, the gneiss occurs as inclusions in the diorite rock mass. The gneiss is thinly layered with quartzite and some schist and is strongly foliated along a northeasterly to easterly bearing and a steep southerly dip. In the past 2 to 3 million years, the dioritic/gneissic rock mass has been uplifted, folded, faulted, and actively eroded; consequently, much of the completely weathered rock has been removed from the steeper slopes. Surficially, most rock on the flatter slopes is completely weathered to a granular, yellowish-brown residual soil, but is only slightly weathered on the steeper slopes. Most outcrops and rock exposed in access road-cuts are slightly to moderately weathered and moderately hard. Little weathering was noted on the drill core obtained from holes drilled in the upstream and downstream portal areas. Weathering extends to greater depths along joints, but has not appreciably penetrated the rock bounded by these joints. The rock is micro-fractured and has relatively low strength. The tested rock strength is discussed in paragraph 8.6.c, and the rock mass classification is evaluated in paragraph 8.6.f.

c. Faults and Joints.

(1) General. The dam site is located between the North and South Branches of the San Andreas fault system. The North Branch fault is about 3,000 feet from the site and the South Branch fault is about 1 mile away. The included area is complexly faulted and jointed. So far only one fault

has been traced across the valley at the site. This fault (Fault A), a possible splay of the North Branch fault, appears to cross the outlet works near the upstream end of the proposed plunge pool. It forms a fault contact between the diorite at the site and the gneissic country rock that the diorite has intruded. This fault and two other minor faults have been recently observed in the Southern California Edison Tunnel by USACE personnel.

(2) Edison Tunnel. Rock conditions in the Edison Tunnel are probably similar to those along the proposed outlet works tunnel. The proposed outlet works tunnel lies downslope about 1,000 feet northwest from the Edison Tunnel at invert El. 2,100, about 220 feet below the invert of the Edison Tunnel, and is roughly parallel with it. The proposed outlet works tunnel will be much larger in diameter compared to the Edison Tunnel, which is only about 7 feet in diameter. Both lie at maximum depths of 450 to 500 feet below the ground surface. Although the Edison Tunnel was constructed in 1882, exposed rock in unlined portions of the tunnel, including the aforementioned fault zones, shows no sign of deterioration or caving. About 35 percent of the 1,400-foot-long tunnel is lined in four sections ranging in length from about 50 feet to nearly 200 feet. Rock conditions in these sections are unknown, but these sections evidently were less stable. There appears to be no support other than the concrete lining. If the lined areas in the Edison Tunnel are fault zones, similar zones probably exist along the outlet tunnel alignment. Most rock in the tunnel is highly jointed at various orientations. A high-angled joint set parallel to the tunnel axis and a low-angled joint set striking nearly normal to that axis are the most notable. Joint spacing varies from 0.5 inches to 4 inches. The close spacing and diverse orientation of the joints forms a very blocky rock mass. Many joints have tight apertures and irregular surfaces. Joint data from the Edison Tunnel appears to be consistent with other joint data observed in field reconnaissance studies and exploratory drilling, and with the joint parameters used in the Rock Mass Rating System (RMR) system (paragraph 8.6.f).

(3) Field Reconnaissance Studies. Fault and joint orientations have also been recorded during other field reconnaissance studies in recent years by USACE personnel and contracted architect-engineers. These studies show that the major joint and foliation planes in rock outcrops in the area of the outlet works generally strike in a northeast-southwest direction and dip at a 75 degree angle to the southeast. The spacing of the joints was found to vary from a few inches to several feet. Some joints contained gouge seams weathered to clayey material. Numerous slickensided shear planes were also observed. The slickensides are probably due to movement that has occurred either as a result of localized stress relief or minor adjustments related to movements on the adjacent segments of the San Andreas Fault. Three other joint systems that were mapped near the outlet structures had strikes of N 15° E, N 35° W, and N 75° W, and dips of 75° NW, 65° NE, and 65° SW, respectively. Notably absent from these studies was the low-angled joint set that was observed along the Edison Tunnel and in the drill core.

(4) Exploratory Drill Cores. All exploratory drilling encountered diorite, but no gneiss; however, prior drilling in the dam foundation did encounter some gneiss. The core from all holes exhibited the same highly jointed rock mass that was observed in the Edison Tunnel and in outcrops, but no major faults. Joint spacing in the drill core can be measured in inches. Joints vary from tight to open, smooth to rough, and planar to slightly wavy. Some joints are clean, whereas others are coated with white siliceous or calcareous material or filled with gray, green, or red clay or reddish gouge. Joint dips reported on the logs range from 0 to 90 degrees, but those around 50 degrees are the most dominant, followed by those in the 30, 40, and 65 degree ranges. The high angled sets observed in the Edison Tunnel and in outcrops were encountered less often in the drill holes because the holes either paralleled the high angled joints or acutely intersected the joints at infrequent intervals.

d. Ground Water and Permeability. No seepage areas have been observed on the left abutment. In the Edison Tunnel most joints were found to be tight and dry except for a few damp surfaces. Holes drilled in the upstream and downstream portal areas also show a preponderance of

tight joints in bedrock. Water pressure tests performed in rock in drill hole C86-30 at the intake structure site gave permeability factors of 0.04 and 0.01 feet per day. At the downstream portal location, a water pressure test in rock in drill hole C86-24 shows a permeability factor of 0.004 feet per day. The average value for all representative permeability values is 0.024 feet per day for all 65 holes that have been drilled in the diorite on both abutments and in the valley section; however, conservatively, a higher value of 0.05 to 0.1 feet per day is assumed for design. At the intake site, ground-water levels in drill holes R86-6, C86-30, and C86-31 were at Els. 2,093.5, 2,098.5, and 2,104.6, respectively. The levels will undoubtedly fluctuate during wet and dry periods. Proposed excavation for the intake structure will be to El. 2,080 and should encounter some ground water in the alluvial material from 23 to 43 feet below the valley floor in this area. These ground-water depths correspond closely to the 10- to 30-foot depths previously encountered during exploratory drilling in the valley for the embankment structure. At the downstream portal, drill hole C86-12 encountered ground water at El. 2,058, 6 feet below excavation sub-grade. In the plunge pool area, seismic refraction line 84-1/2 indicates that saturated alluvium may be as shallow as 15 to 20 feet below the ground surface.

8.6 Design Criteria and Assumptions.

a. Basic Data. Bedrock is assumed to be dioritic except for some scattered inclusions of gneiss. All rock is moderately hard to hard, and highly fractured. Weathering is not excessive; some slight weathering can be expected at all depths, especially along fractures. The rock quality does not improve substantially with depth.

b. Rock Quality Parameter Comparisons. The results of the RQD and core recovery percentage calculations for each core run are presented on the individual core logs on plates 8-2 and 8-3. Individual point load strength index results are also depicted on the core logs along with interval permeabilities as determined from packer tests. In addition, the RQD and point load strength values are presented graphically on plate 8-4 along with a plot of discontinuity frequency and a qualitative depiction

of the degree of weathering. These four parameters were chosen to represent the overall rock quality of the cores. From the graphs, it is possible to identify zones of poor quality rock (low RQD, high discontinuity frequency, low strength, and high degree of weathering) especially relative to the average values for each individual hole. An overall summary of the rock quality parameters derived from rotary and core drilling is presented below.

Inlet Portal

Hole No.	Rock Type	Bedrock Depth (feet)	Recovery (Percent)	RQD (Percent)	Average Discont. Freq. (per ft)	Average Point Load (psi)	K (ft/day)
R86-06	Diorite(?)	70.0					
C86-07	Diorite	36.5	97	38	3.1	339	
C86-30	Diorite	74.5	98	46	5.4	176	0.03
R86-31	Diorite(?)	87.5					
	Overall Averages:		98	42	4.3	258	0.03

Outlet Portal

Hole No.	Rock Type	Bedrock Depth (feet)	Recovery (Percent)	RQD (Percent)	Discont. Freq. (per ft)	Point Load (psi)	K (ft/day)
C86-12	Diorite	67.5	88	28	3.9	265	
C86-13	Diorite	60.6	97	38	3.3	60	
C86-24*	Diorite	33.5	98	36	5.1	300	0.01
C86-25	Diorite	41.6	100	32	4.4	266	
Overall Averages:		56.6	96	34	4.2	223	0.01

*Indicates inclined hole.

Energy Dissipator

Hole No.	Rock Type	Bedrock Depth (feet)	Recovery (Percent)	RQD (Percent)	Discont. Freq. (per ft)	Point Load (psi)	K (ft/ day)
R86-06		68.5+	(bedrock not encountered)				
C86-10	Gneiss?	121.5(?)	16	0	10.0		

c. Laboratory Test Results. The results of laboratory tests conducted on samples from the outlet works are presented in the following paragraphs. Unconfined compression tests were conducted on two core samples from the inlet portal area. The results of those tests are presented below.

Outlet Works - Inlet Portal
Results of Compressive Strength Tests

Hole No.	Rock Type	Depth (feet)	Unconfined Compressive Strength (psi)	Point Load Index (psi)	Ratio
C86-07	Diorite	84.9	7,884		
C86-30	Diorite	115.2	9,889	315	31.4

The sample from hole C86-07 was also tested for specific gravity and absorption with the following results:

Hole No.	Rock Type	Depth (feet)	Bulk SSD	Absorption (percent)
C86-29	Gneiss	77.1 - 77.8	2.75	0.71
C86-07	Diorite	84.9 - 85.3	2.73	0.87

A specific gravity of 2.73 equates to a unit weight of 170 pounds per cubic foot.

d. Rock Quality Designation. The rock quality designation (RQD) is a modified core recovery classification to define the rock quality by a percent and is a principal variable in evaluating rock behavior around a tunnel. The classification is determined by measuring the length of each hard and sound drill core (soft altered cores are not counted) which is longer than 4 inches, about twice the core diameter (originally based upon an NX or 2 1/8-inch-diameter core). The accumulative length of these cores is divided by the total length of drill run under study to obtain the percent RQD. If the core is broken by handling or by the drilling process (i.e., the core ends are fresh breaks rather than being bounded by natural joint surfaces), the fresh broken pieces are fitted together and counted as one piece, provided that they form the required length of 4 inches or more. In holes where mechanical problems or poor drilling techniques create core loss zones, the percent RQD may become unduly low. Based upon the holes drilled in the diorite rock mass at the dam site, including six holes drilled at the intake and outlet portal areas, the average RQD has been determined to be about 36 percent. The average joint/discontinuity spacing is about 3 inches.

e. Rock Strength and Deformation.

(1) Compressive Strength. Unconfined compressive strength tests were performed on 17 intact rock core specimens from 10 holes drilled in the dioritic rock mass across the dam site area. The unconfined compressive strengths of the specimens tested range from 1,790 psi to 13,180 psi. The average strength was 5,636 psi. The average strength of three samples from holes in the upstream portal area was 7,316 psi. All samples represent typical foundation rock. All samples exhibit moderate to severe alteration and weathering of the feldspar minerals and mafic constituents. The greatest strengths were achieved in those samples where the feldspars were relatively fresh, biotite, and other mafic minerals were randomly oriented and no filled fractures were present. Assuming that the rock quality does not improve with depth, an intact rock strength of 5,000 psi is considered appropriate for preliminary foundation design of the outlet works.

(2) Point Load Strength. Forty-three point load tests were performed on cores from six drill holes in the intake and outlet structure areas. The point load test is a strength index test that can be performed in the field quickly and cheaply on drill core without the need of machined or special prepared samples. It is conducted by applying a compressive point load diametrically to the core specimen and measuring the load at failure. The core actually breaks in tension. The point load strength index (Is) is derived by dividing the failure load by the square of the core diameter. Point load strength is closely correlated to the results of uniaxial compression and other strength tests. The approximate uniaxial compressive strength is equal to about 24 x Is for HQ (60 mm diameter) core (reference figure 5, Rock Testing Handbook [RTH] 325-82 for conversion factor). The average point load strength for the cores at the inlet is 238 psi and 253 psi at the outlet. This index strength value converts to approximately 5,660 psi at the inlet and 6,020 psi at the outlet which is slightly higher than the average unconfined compressive strength obtained in the laboratory.

(3) Allowable Bearing Capacity. Literature values for allowable bearing capacity for fresh igneous and metamorphic rocks, similar to those at the Seven Oaks dam site, are as high as 5 to 6 mega pascals (approximately 800 pounds per square inch [psi] or about 60 tons per square foot [tsf]). Considering that the rock at the site is both highly fractured and partially weathered, this value should be reduced about 50 percent. Hence, 30 tsf is a reasonable value for the allowable bearing capacity of the rock at the site. A very similar value can be obtained when a common rule-of-thumb method is used. In this method, the bearing capacity is equal to about one-fifth of the unconfined compressive strength. One-fifth of 4,000 psi is also 800 psi or 60 tsf. Since the unconfined compressive strength is determined with intact rock cores, a similar reduction in value to 30 tsf is in order. For conservative design purposes, however, the assumed allowable bearing capacity value was further reduced to 20 tsf.

(4) Cohesion and Shear Resistance. Since no direct or triaxial shear tests have been performed on diorite rock from Seven Oaks, shear strength parameters have been determined by empirical analysis. Shear strength parameters for contacts between foundation rock and concrete are assumed to have cohesion values of 100 psi (14.4 kips per square foot [ksf]) and phi angles of 35 to 40 degrees. Shear strength parameters for joints and shear zones that may be continuous and capable of producing wedge failures in excavation slopes and in the tunnel crown will be assumed to have a 0 psi cohesive strength and phi angles of 10 to 20 degrees.

(5) Deformation. Stresses imposed on relative large rock masses by engineering structures, such as the intake tower, will cause the rock to deform or compress due to discontinuities in the rock. The deformation modulus is a measure of this compressibility. It can be measured most accurately by large-scale field load tests, but it can also be estimated from laboratory tests on intact core specimens. The elastic modulus obtained from an intact core specimen, though, is always larger than that obtained from a field test. When the joints are widely spaced and very tight in a rock mass, Young's modulus (the modulus of elasticity) establishes the upper limit for the deformation modulus, but as the rock mass becomes more intensely jointed, the deformation modulus is reduced to a smaller fraction of Young's modulus. The ratio of the deformation modulus measured from field tests to the deformation modulus measured in the laboratory on intact specimens is defined as the reduction factor. Stagg and Zienkiewicz (1968), in "Rock Mechanics in Engineering Practice," show a correlation between reduction factors and RQD values. Since the reduction factor for rock with an RQD of less than 60 percent is about 0.13, the deformation modulus for the rock at Seven Oaks (with an RQD of 35 to 40 percent) would then be not greater than 0.13 x 2 x 10^6 psi or 0.3 x 10^6 psi. According to Stagg and Zienkiewicz, 2 x 10^6 psi is the average value of Young's modulus for intact igneous rock at an unconfined compressive strength (UCS) of 5,000 psi. In comparison, Bieniawski (1979), in "Tunnel Design by Rock Mass Classification," shows a relationship between the in situ modulus of deformation and the rock mass rating (reference RMR discussion in paragraph 8.6.f). This relationship shows that rock with a 0.3 x 10^6 psi modulus should have an RMR of 60.

The rock at Seven Oaks has an RMR of 50 or less, depending upon the interpretation of joint and ground-water conditions. This rating, though, could possibly be as high as 60 if evaluation of the rock mass quality is restricted to samples from locations at or below foundation grade along the outlet alignment. More field and laboratory data on the rock is needed. For example, the RMR of 60 would require UCS and RQD values of 8,000 psi and 50 to 60 percent, respectively, which is not unrealistic. No testing for the modulus of deformation or elasticity has been performed on rock from the Seven Oaks site. Typical modulus values for different rock types, though, can be obtained from literary sources. A suggested value for the deformation modulus for unweathered igneous or metamorphic rock masses by Bazant (1979), in "Methods of Foundation Engineering," is 5×10^6 kN/m^2 (0.725×10^6 psi); however, our recommended value for the modulus of deformation is 0.3×10^6 psi since the rock parameters used to determine Bazant's value are unknown.

f. Rock Mass Classification.

(1) General. Most tunnels are designed with the aid of a rock classification system which utilizes an empirical approach as compared to an analytical or observational approach. An analytical approach is seldom used because of insufficient input data, and an observational approach such as the New Austrian Tunneling Method, which uses a "build as you go" philosophy, is not as feasible in this country due to less flexible contracting procedures. For this report, the preliminary tunnel design is based primarily on two rock classification systems, the old Terzaghi Classification and the more recent Rock Mass Rating System (RMR) developed by Bieniawski, and on general experience from previous tunnel projects.

(2) Terzaghi. The Terzaghi Classification is used to determine rock loads and to select the appropriate spacing and size of steel sets for tunnel support. Under this classification the rock for the outlet works tunnel was considered to be Class 5 (very blocky and seamy rock with little or no side pressure), and the rock load (Hp) was selected as 0.5(B+H)Wr where B is the tunnel width, H is the tunnel height, and Wr is the rock unit weight.

(3) RMR. The RMR System utilizes the following parameters:

(a) Uniaxial compressive strength of intact rock.

(b) Rock quality designation.

(c) Spacing of joints.

(d) Orientation of joints.

(e) Condition of joints.

(f) Ground-water conditions.

Implementation of these parameters in the RMR system is discussed in detail in Technical Report GL-79-19 by WES. In summary, though, the parameters are rated numerically by importance. These ratings are then summed and given an overall rock mass rating which is then assigned to one of five rock mass classes. The meaning of the rock mass classes has been formulated into standard excavation and support parameters, such as standup time, allowable unsupported rock spans, rock mass cohesion, and the angle of internal friction. Excavation procedures and support requirements have also been predetermined for each class. Using data from paragraphs 8.5 and 8.6 for the rock mass parameters, the rock mass along the entire outlet works tunnel is classified as class IV, which is a poor rock; however, since some of the values for the rock parameters are conservative, a class III (fair rock) would not be unreasonable to use at this stage of investigation and design. Excavation procedures and support requirements for class IV rock are given in paragraph 8.7.d.

g. Determination of Seismic Design Criteria. Seismic design parameters for the Seven Oaks dam site were determined by Dr. Bruce A. Bolt (reference section 7, "Structural Design"). The seismicity of the region stems from the numerous major faults that exist within approximately a 50-mile radius of the dam site. The two major faults that are in closest proximity to the dam site, however, determine the seismic design

criteria. The west-northwest trending North Branch of the San Andreas Fault lies approximately 3,000 feet north of the proposed dam centerline and transects the reservoir area. Approximately 1 mile south of the dam site at the mouth of the Santa Ana River Canyon, lies the South Branch of the San Andreas Fault. The South Branch is recognized to be the plane of most recent movement. Based upon the work of Professor Kerry Sieh, California Institute of Technology, there is a 50 to 90 percent probability of a great earthquake on the South Branch of the San Andreas Fault (although not necessarily in the vicinity of the dam site) within the next 50 years. This earthquake (maximum probable earthquake) would have an estimated magnitude (Ms) between 7.5 and 8.0. For an assumed source distance of 20 miles, the estimated peak horizontal acceleration at the site is 0.5g. The largest conceivable earthquake (maximum credible earthquake) for this vicnity would have an Ms of 8+ and, at an assumed hypocentral distance of 2 miles, would result in an estimated peak horizontal acceleration of 0.7g at the site. The higher (MCE) criteria will be used for seismic design (see Section 7).

8.7 Excavation and Treatment.

a. Approach Channel and Plunge Pool. The upstream diversion approach channel from about Station -1+00 to Station 9+75 and the downstream plunge pool will be excavated in alluvial valley-fill. Mechanical equipment will be used to excavate the alluvium except that some light shooting may be required to break up large boulders. The alluvium at both locations will be excavated to 1V on 2H slopes. The approach channel excavation should encounter bedrock near Station 9+75, and should be almost entirely in rock at Station 10+74 (upstream edge of intake structure). All rock will be drilled and blasted to 4V on 1H slopes. The approach channel alignment has been designed to simulate the same hydraulic gradient that occurs along the natural stream channel. This gradient is approximately 3 percent from El. 2,100 at the intake structure to El. 2,130 in the streambed about 1,040 feet upstream. Dewatering should be minimal in the approach channel excavation except below El. 2,105 near the intake structure, however, increased dewatering may be required during rainy periods. Dewatering will also be required below El. 2,000 in the plunge

pool excavation. The average transmissivity of the streambed alluvium has been previously reported at 133,000 gal/day/ft^2. The excavations will require no special treatment.

b. Intake Structure and Portal. Excavation for the intake structure will be mostly in rock. Alluvium up to 10 feet deep will probably be encountered in the right upstream corner of the excavation. All bedrock will be excavated by systematic drilling and blasting. The intake structure will be founded on rock at El. 2,080. Alluvial deposits will be laid back to stable slopes or stripped to rock. Rock cut slopes, which will be covered by concrete on the sides of the structure below El. 2,156, will be near vertical. For estimating purposes, these slopes were assumed to be 10V on 1H which eliminates possible overhangs and compensates for necessary "step-outs" of blast holes at the back of each succeeding lift. All other rock cut slopes in the intake structure will be 4V on 1H. All rock cut slopes will be rock bolted and shotcreted as conditions require as the excavation proceeds downward. All slopes will be scaled and then cleaned if shotcrete is to be applied. Rock bolts will be installed after shotcreting although some initial spot bolting may be required. All cleanups will be performed by jetting with compressed air and/or water jets, limited to a maximum pressure of 100 psi at the discharge nozzle. Weak, weathered, or altered zones in rock at foundation grade that continue to have loose, unkeyed rock fragments and/or finer material during jetting, and that are too narrow to excavate with mechanical equipment, will be excavated with hand, pneumatic tools to a depth of three times the width of the zone, then backfilled with dental concrete. Although the estimated quantities reflect an 8-foot by 8-foot staggered rock bolt pattern with alternating 15-, 20-, and 25-foot-long bolts, some areas may not require bolts. Other areas may require longer or shorter bolts, depending upon results from future wedge-failure analyses as sufficient joint data becomes available. Prior to turning under, closely-spaced patterned bolts will be required outside the tunnel periphery to stabilize the portal and control overbreak. Rock bolts will be 1 1/4-inch-diameter, resin grouted, and tensioned. The estimated quantities also reflect a 2-inch-thick layer of fiber-reinforced shotcrete over the entire rock-excavated surface above invert grade, including a

doubling factor which compensates for irregularities in the rock surface and waste due to rebound. No mesh will be required. Safety rock barrier fence will be required around the perimeter of the high-cut rock excavation. Dewatering may be required below El. 2,100.

c. Downstream Portal. Location of the downstream portal was determined by analyzing the potential arch-action or overbreak of the rock in the tunnel crown. A large portal in very blocky and seamy rock (by the Terzaghi Classification System) could feasibly fail before adequate support is installed. This failure could form a natural arch to a height of 0.5 (H + B), above the crown. The rock cover above the tunnel crown at the portal, therefore, should be 1 1/2 to 2 times this height so a roof can be formed over the arch to prevent complete failure of the overlying rock wedge. The shield tunneling method could be used with less rock cover during excavation, but higher costs associated with this method probably would not offset any rock/soil excavation savings. Some reduction in excavation costs could be realized if the berm was eliminated and the portal was adjusted slightly downstream. All portal rock slopes will be 4V on 1H. A 20-foot-wide rock berm above the portal at El. 2,130 will be an impeder to falling rock and other material sloughing into the exit channel. All rock will be excavated by systematic drilling and blasting, and all overburden will be removed with mechanical equipment. All overburden slopes will be excavated 1V on 2H, but stripped to rock where the underlying rock line is steeper than 1V on 2H above the portal area. All rock slopes will be scaled and all rock surfaces receiving shotcrete or concrete will be jet-washed. All rock slopes will be rock bolted, shotcreted, and fenced in the same manner as outlined for the intake structure and portal areas. Since ground-water depths are below foundation grade, dewatering of the excavation should not be required.

d. Tunnel/Shaft(s).

(1) Excavation Procedures. The tunnel (and shafts, if required) will be driven mostly through a highly fractured diorite rock mass and possibly some gneiss, as discussed in paragraph 8.5. On the basis of rock

structure, no preferred tunnel advance direction is foreseen; however, an upstream advance would protect the tunnel from floods and minimize the haul distance, provided that downstream disposition of the tunnel muck is preferred. Also, an upstream advance would provide better drainage if ground water is encountered in any substantial amounts. For the mid-tunnel control alternative, an advance from both directions is assumed to maintain the same schedule. The tunnel excavation will be horseshoe-shaped. Excavation will be by drilling and blasting using either the full-heading or heading-and-bench method depending mainly on the tunnel cross section of the selected alternative. Since the rock is highly fractured, a short standup time is expected (5 hours for a 5-foot span for class IV rock by the RMR rating system). Tunnel advancement with short tunnel rounds of 3 to 5 feet is implied by the RMR analysis, but 5- to 10-foot rounds may be possible. Shaft excavation (mid-tunnel control alternative) would probably be advanced by the raise bore method; excavation of the oversize tunnel section to house the control gates would be accomplished by slashing to the final dimensions under a carefully controlled sequence of excavation and support installation as shown on plate 8-6.

(2) Rock Loading, Tunnel/Shaft Support, and Reinforcement. In accordance with class IV rock by the RMR system, the outlet works tunnel may require:

(a) Light to medium steel rib sets spaced 4 feet apart.

(b) Four- to 6-inch-thick shotcrete in the roof and 4-inch-thick shotcrete on the walls (probably 2- to 3-inch thickness for fiber-reinforced shotcrete).

(c) Installation of rock bolts 12 to 15 feet long, spaced 3 to 5 feet in the roof and walls.

(d) Installation of support concurrently with the excavation.

Typical final rock load on the roof support in the tunnel (depending on the alternative selected) using the Terzaghi Classification, as discussed in paragraph 8.6.f, could range from approximately:

$$
\begin{aligned}
Hp &= 0.5(B+H)Wr \\
&= 0.5(20\text{ ft} + 23.5\text{ ft})\ 170\text{ lbs/ft}^3 \\
&= 3{,}700\text{ lbs/ft}^2
\end{aligned}
$$

to approximately:

$$
\begin{aligned}
Hp &= 0.5(B+H)Wr \\
&= 0.5(22\text{ ft} + 36\text{ ft})\ 170\text{ lbs/ft}^3 \\
&= 4{,}930\text{ lbs/ft}^2
\end{aligned}
$$

Steel rib sets on 4-foot centers with continuous shotcrete blocking and lagging are anticipated along most of the tunnel. Spot bolting and spiling would also be used as necessary. Higher loads in larger diameter sections of the tunnel will require heavier ribs or a combination of fiber-reinforced shotcrete and pattern rock bolts. Shaft support (mid-tunnel control) would probably consist of a 4- to 6-inch layer of fiber-reinforced shotcrete, and possibly pattern rock bolts, applied at the completion of the raise bore.

(3) Grouting.

(a) Contact. The crown of the outlet works tunnel will be contact grouted along the entire tunnel length. Grouting will be performed every 10 feet of tunnel length through nipples of 2-inch-diameter black steel pipe embedded in the concrete lining at high excavation points. Vent pipes should also be installed to eliminate air pockets and to assure positive grout control. During grouting, grout pressures will not exceed the strength of the concrete liner. After grouting is completed, all protruding pipe will be cut off, and the liner will be patched and ground smooth.

(b) Consolidation. To reduce potential seepage along the tunnel bore, consolidation grouting will be used to cement broken or fractured rock around the upstream tunnel portal that has been loosened by excavation and stress relief. The grout hole pattern will consist of one or more rings of holes around the periphery of the tunnel portal. The hole spacing and depth will depend upon the degree or extent of fracturing after excavation. Grouting will consist of pumping neat portland cement grout through nipples of 2-inch-diameter black steel pipe installed in 1 1/2-inch (EW) holes. Grouting will be by the split-spacing, stage-grouting method. Grout pressures will not exceed an effective pressure of 1 psi per foot of hole depth. Water-pressure tests should be performed in all holes prior to grouting to assess permeability characteristics and water/cement ratios.

(c) Ring. To further reduce the potential of seepage along the tunnel bore, a grout ring curtain will be constructed around the completed tunnel about midway between transverse joints in the concrete liner beneath the dam grout curtain. The ring curtain will consist of one ring of 1/8-inch-diameter holes, drilled and grouted in one 20-foot zone by the split-spacing, stage-grouting method. Crown holes may be extended to the dam grout curtain. The holes will be drilled every 40 degrees of arc in a radial direction. All grouting will be done through embedded pipes or formed holes in the concrete liner. No grouting will be done until 7 days after placement of the liner. Grout pressures will not exceed the strength of the concrete tunnel liner. Water-pressure tests will be performed in all holes prior to grouting to assess permeability characteristics and permissible water/cement ratios. After grouting is completed, all protruding pipe will be cut off, and the liner will be patched and ground smooth.

(4) Hydrostatic Gradient and Loading. General guidance for determining the thickness of tunnel linings due to external water loads on non-pressurized tunnels is cited in paragraph 3-7b(3) and Appendix H of EM 1110-2-2901, "Tunnels and Shafts in Rock," but specific criteria on how to determine the proper hydrostatic head and gradient, water loads, and drainage methods to reduce loading are not discussed. Some general concepts can be found in Section 3.7 of K. Szechy's book on "The Art of

Tunnelling," and Section D-241 in the "Design of Small Dams" by the Bureau of Reclamation. Although several different reservoir storage levels could be selected for the hydrostatic head at Seven Oaks, the debris pool (El. 2,300) is the recommended level since it not only occurs most frequently (about every 2 years), but it also has the longest duration (about 100 days) as shown in figure 7-2. At this head, the tunnel liner should be designed to a safety factor of 2, but likewise, this design should be able to sustain the maximum hydrostatic load at standard project flood level (El. 2,572) with a safety factor of not less than 1. The loading factors used in figure 7-3 and discussed in section 7 meet this criteria. The typically sloping hydrostatic gradient, normally used for analysis of this kind, has been stair-stepped to best reflect the rapid loss of seepage pressure in rock of low permeability at an estimated distance of 230 feet from the intake portal. The hydrostatic pressure is assumed to be non-existent downstream from the ring grout curtain. External hydrostatic pressures along the tunnel will be reduced by a factor of 50 percent or more by installing drains as discussed in the following paragraph.

(5) Drainage. A drainage system will be required along the outlet tunnel for a distance of approximately 100 feet downstream from the portal. Water loads on the tunnel liner can be reduced to 50 percent or more with a drainage system. Existing data indicates that water pressures along the outlet tunnel will not be sufficient to cause failure of the liner beyond this distance because hydrostatic pressures normally diminish with distance along seepage paths, especially in tight-jointed rock. The coefficient of the rock mass along the tunnel varies from 0.05 to 0.1 ft/day. If excessive pressures are subsequently encountered in other parts of the tunnel, including overexcavated shear/fault zones, additional drains will be provided. The drain system design will consist of nearly semicircular, drain-hole rings installed on 20-foot centers. Each drain-hole ring will consist of four 3-inch-diameter holes drilled radially from the tunnel crown every 40 degrees of arc. Drains will not be constructed until all tunnel grouting is completed. Embedded or external plumbing will be required to drain intercepted water into the bottom of the tunnel.

(6) Shear/Fault Treatment. Studies of tunnel damage from the 1952 earthquake of magnitude 7.7 in Kern County, California, showed that only four of 15 tunnels were severely damaged during the earthquake (Elementary Seismology by Charles F. Richter, 1958). The four tunnels were broken and offset by thrusting on the White Wolf fault which crossed the tunnel alignments. There were no collapses of tunnels that experienced damage by displacements. In Dr. Clarence Allen's assessment of the fault rupture hazard at the Seven Oaks dam site (Supplement to Phase I GDM, December 1986), he indicated that a design displacement of as much as 4 feet in any direction should be assumed. For the downstream control design alternative, the tunnel would be overexcavated along the entire length to allow for 3 feet 5 inches of clearance around a steel pipe (2 feet, 1 inch at rib stiffener locations) placed inside a concrete-lined, horseshoe-shaped tunnel. This design would provide space for the repair of the tunnel and allow the pipe to be brought back to the original line and grade after displacement occurs on the fault plane. This defensive measure would not be possible, however, for the other two control alternatives.

(7) Instrumentation. The tunnel at Seven Oaks should be instrumented during construction to verify design assumptions and to monitor deformation of tunnel walls and crown, and load on the steel sets. The following instruments are suggested at this design stage:

(a) Multiposition extensometers with a maximum penetration depth of 50 feet should be installed in rock in an array of three (one on each side and one in the crown) at two stations about 800 feet apart.

(b) Tape extensometers should be installed approximately every 50 feet.

(c) Selected steel sets should be monitored by installing strain gauges on selected ribs. Strain gauges should also be installed on selected ribs in major shear zones.

8.8 Intake Structure Bridge and Access Road.

a. Alignment and Design Criteria. The access road to the intake structure will start at the top of the embankment dam (El. 2,610) on the right abutment, and will run across the upstream face of the dam to the intake structure bridge (El. 2,299) on the left abutment, a drop of 311 feet along 4,800 feet of road. The road gradient will vary from 2 to 10 percent, but will average about 6 percent. About 2,400 feet of road will be built on fill placed on the embankment face; the remaining 2,400 feet of road will be cut into rock, primarily on the left abutment starting at the intake structure bridge abutment. The roadway will be 15 feet wide, single lane with shoulders and turnouts, and paved. The subgrade material will be about 1 foot thick minimum. The road will be used for general access, project crane, and dump truck operations.

b. Bridge Abutment and Rock Excavation Sections. The intake structure will be accessed from the road by a 54-foot span, single-lane bridge at El. 2,299. The bridge will extend downstream from the structure along the outlet works alignment and abut the upper slope of the intake structure excavation to the left of centerline and into the natural rock slope on the right side. The bridge abutment footings will require a rock bench near El. 2,285. That bench will be about 25 feet wide and will extend a minimal distance of 30 feet into the hillside. All rock slopes for the bench excavation will be 2V on 1H. The road leading from the bridge abutment to the embankment dam will be cut into rock. For estimating purposes, the outside of the cut will daylight at road grade, and the inside cut slope will be 2V on 1H. For final design, however, slopes of 4V on 1H with intermediate berms may be required, as some inside slopes will reach heights between 100 and 200 feet just beyond the bridge abutment around a 100-foot radius curve. This design change should not appreciably change the original quantity estimate. Rock bolts and mesh will be required on most slopes. The rock is expected to be mostly highly fractured diorite with gneissic inclusions.

c. Embankment Road Fill Section. The fill section of road across the upstream face of the embankment dam will consist of sound, angular rock,

free from silt, clay, organic material, and foreign debris. The fill should have a gradation similar to the Zone 3 or 4 material of the dam embankment. The top of the fill will be of sufficient width to accommodate the roadway, shoulders, and turnouts as required. The fill slope will be 2V on 3H as compared to the 1V on 2H slope for the embankment. The fill will be inundated to varying elevations during the life of the project and probably will be subjected to rapid drawdowns.

8.9 Future Work.

a. Field Investigations. Additional explorations along the outlet works alignment will be accomplished to better define:

(1) The top of foundation rock within the excavation limits of the portal excavations.

(2) The piezometric surface along the tunnel alignment.

(3) Permeability values around the tunnel.

(4) Joint attitudes, spacing, and conditions.

(5) RQD.

(6) The rock mass quality in general.

(7) Residual in situ stresses.

(8) Presence of significant shears.

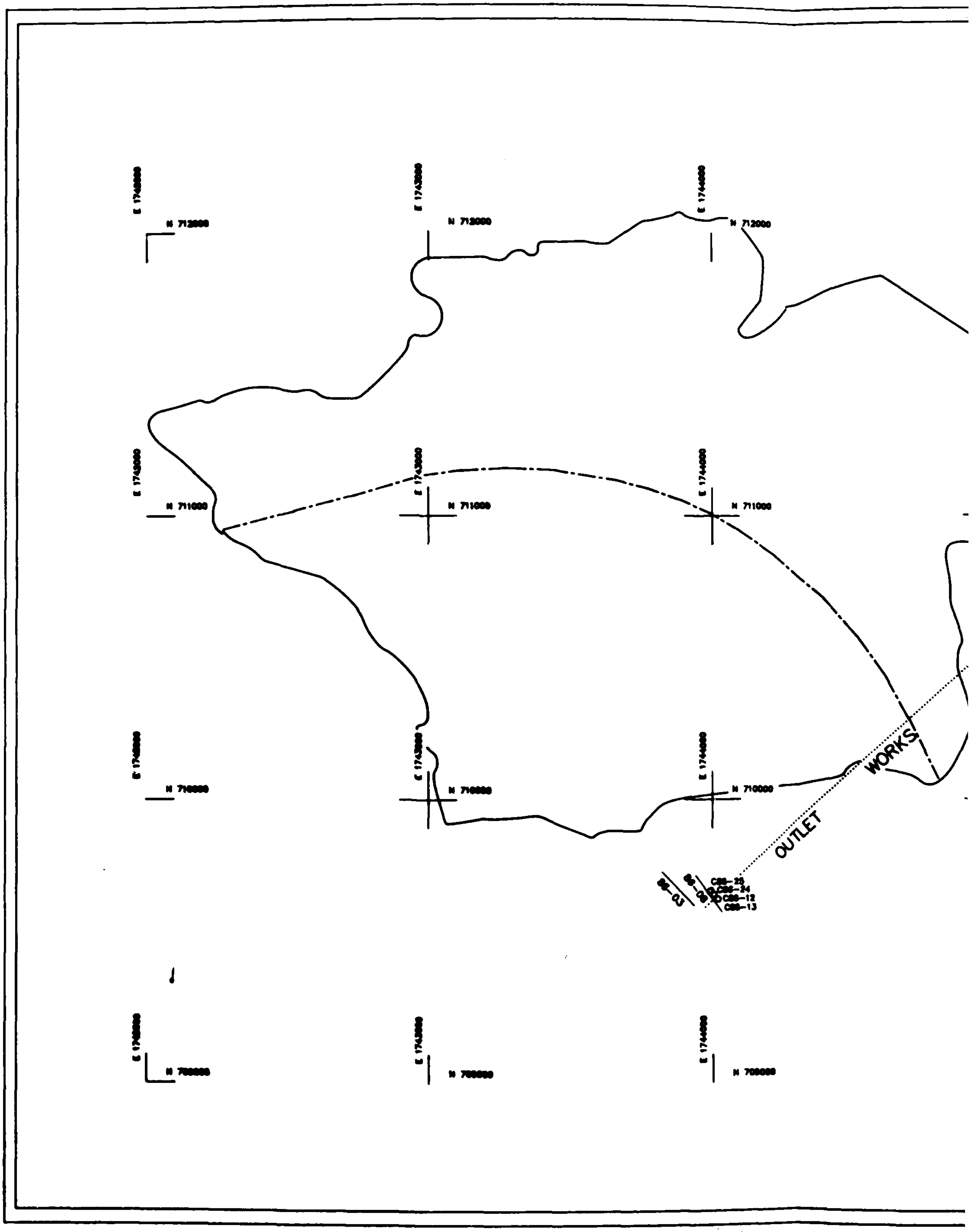
At least one horizontal hole, several hundred feet long, is planned at each portal area to determine characteristics of the steep joint sets and general conditions along the crown of the tunnel. Several deep, vertical and inclined holes will be drilled along the tunnel route where horizontal holes have not penetrated. At least one of these holes is planned in the vicinity of the proposed shaft location(s) for the mid-tunnel control

alternative. Several additional holes are also planned in the portal areas to facilitate excavation layouts with respect to the top of rock contours and to obtain test specimens for strength and modulus values. Piezometers will be installed in most holes and read regularly for seasonal fluctuations of the ground-water level. Joint diagrams by stereographic projection will also be prepared from field surveys of all measurable joints to facilitate joint evaluations for stability studies. Dozer trenching and backhoe excavations along the approach channel are planned to evaluate the excavatability and dewatering characteristics of the alluvial material. Drifts will be excavated into both portals to better define critical rock conditions and fracture orientations, and to conduct residual stress tests.

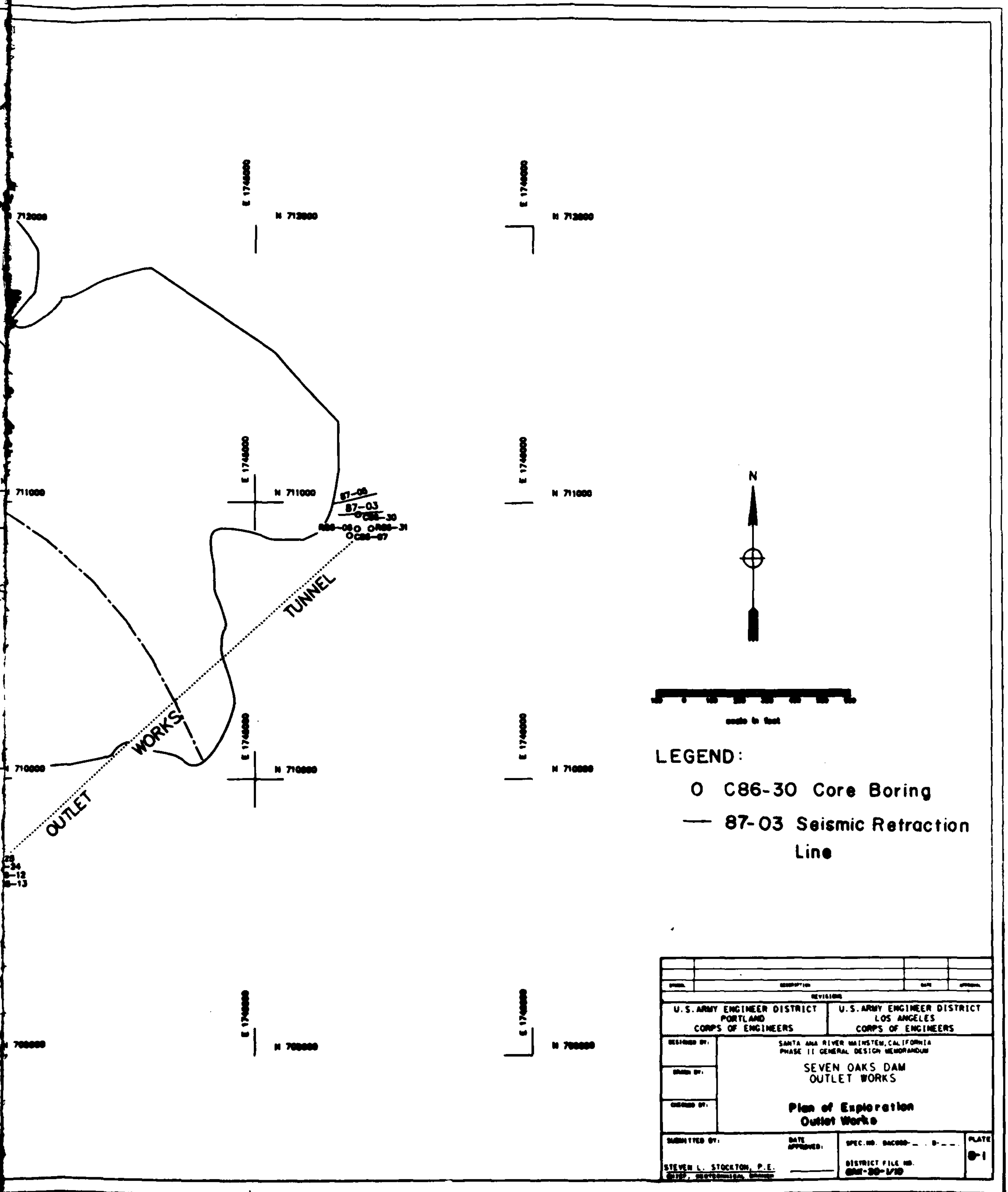
b. Testing and Evaluations. The following laboratory testing is planned for future design work:

(1) Direct shear tests on intact, jointed, sawn, and concrete-bonded cores to establish maximum, residual, and cohesion strength values and pertinent phi angles for sliding stability analysis of foundations, slopes, and rock wedges therein.

(2) Additional unconfined compressive strength tests, including values for Young's Modulus and Poisson's Ratio, especially on intact cores from the foundation area of the intake structure from which deformation characteristics can be determined.



E 1743000
E 1744000
N 712000
N 711000
N 710000
OUTLET
WORKS



E 1748000
N 713000
N 711000
N 710000
N 709000
87-03
C86-30
C86-31
C86-07
OUTLET WORKS TUNNEL
N
scale in feet
LEGEND:
O C86-30 Core Boring
— 87-03 Seismic Retraction Line
REVISIONS
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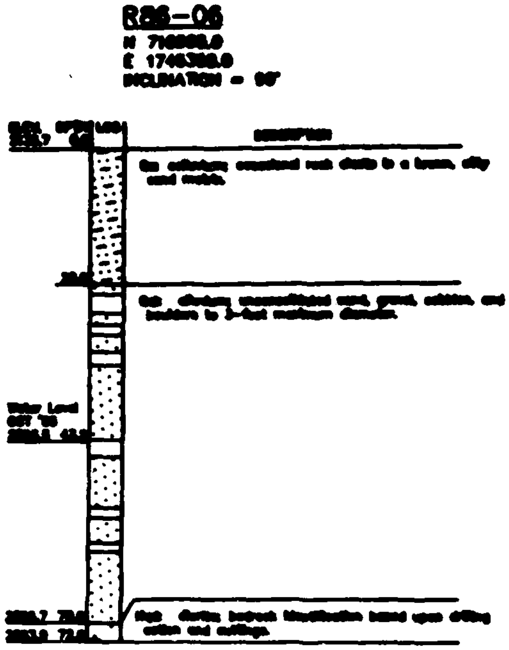
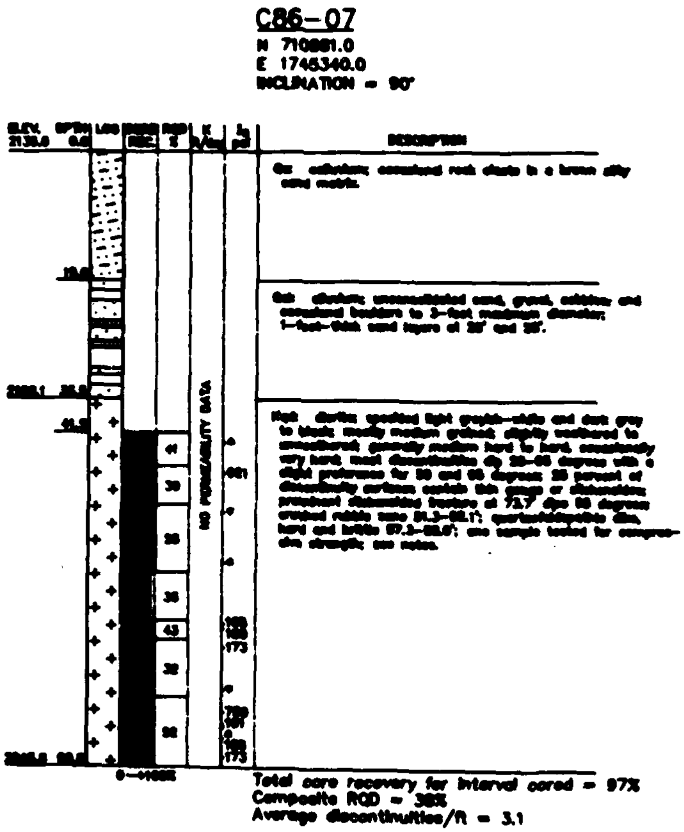
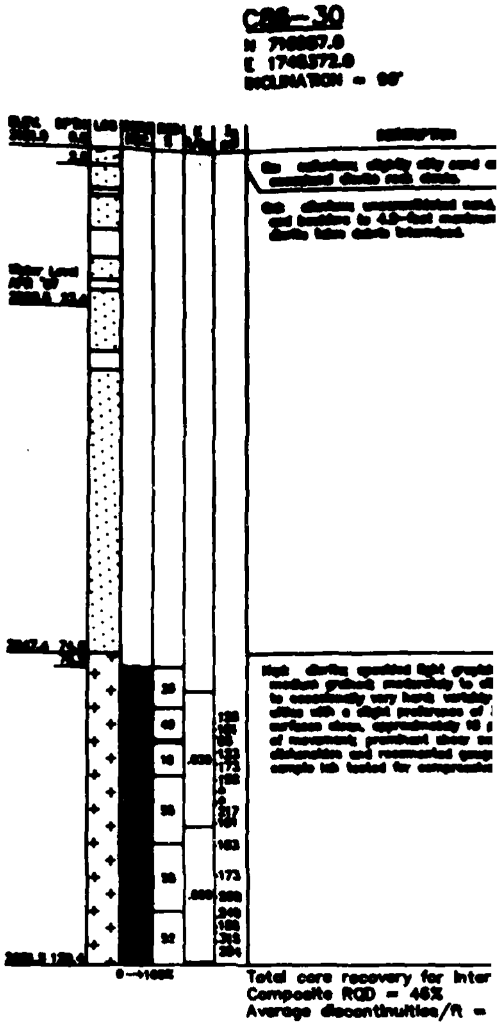
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM
SEVEN OAKS DAM
OUTLET WORKS
Plan of Exploration
Outlet Works
SUBMITTED BY:
STEVEN L. STOCKTON, P.E.
DATE APPROVED:
DISTRICT FILE NO.
PLATE

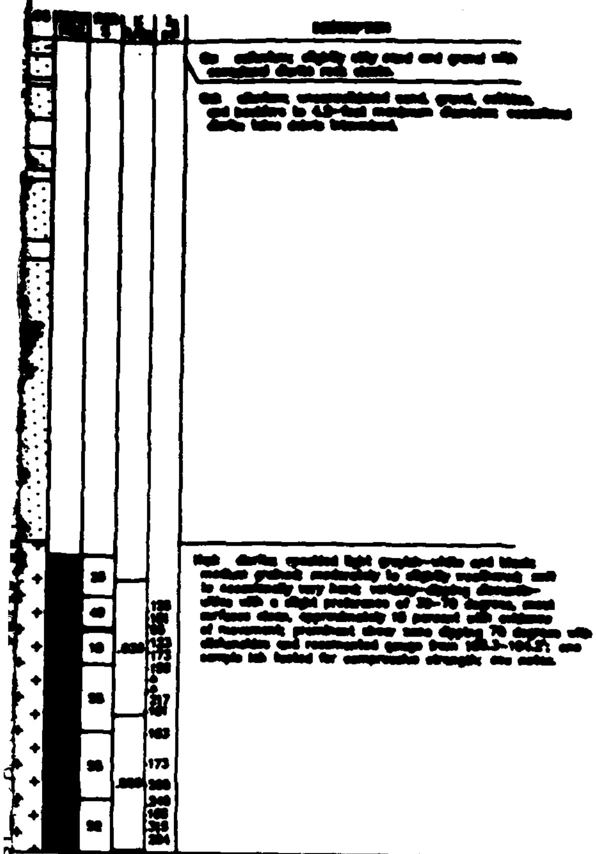

R86-06
INCLINATION = 90°


C86-07
N 710881.0
E 1745340.0
INCLINATION = 90°
ELEV.
2138.0
0.0
LOG
REC.
K
DESCRIPTION
19.0
NO PERMEABILITY DATA
41
38
38
36
43
32
32
.081
0-100%
Total core recovery for interval cored = 97%
Composite RQD = 38%
Average discontinuities/ft = 3.1


C86-30
INCLINATION = 90°
DESCRIPTION
Water Level
0-100%
Total core recovery for inter
Composite RQD = 46%
Average discontinuities/ft =

CB6-30

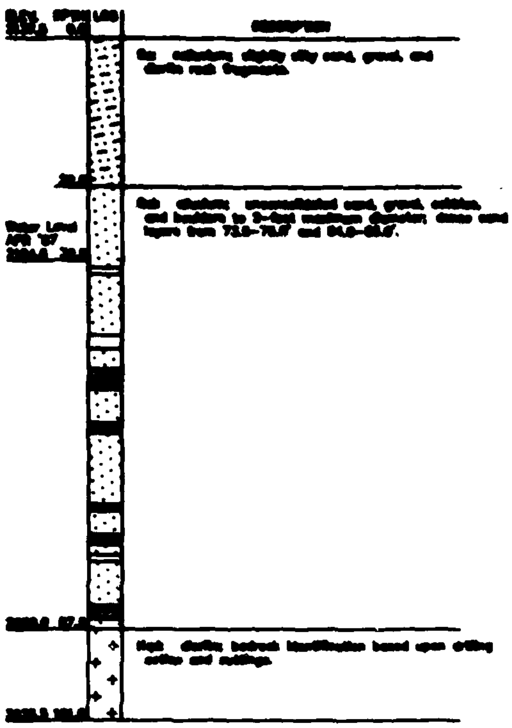
N 7108[illegible].0
E 1748372.0
INCLINATION = 90°



Total core recovery for interval cored = 98%
Composite RQD = 46%
Average discontinuities/ft = 5.4

RB6-31

N 7108[illegible].0
E 17484[illegible].0
INCLINATION = 90°



NOTES:

1. See plate B-1 for locations of core holes.

LITHOLOGIC LEGEND

Overburden:

	Qal	Alluvium
	Qc	Colluvium
		Sand layer
		Boulder

Bedrock:

	Kqd	Diorite
	Gn	Gneiss

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SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

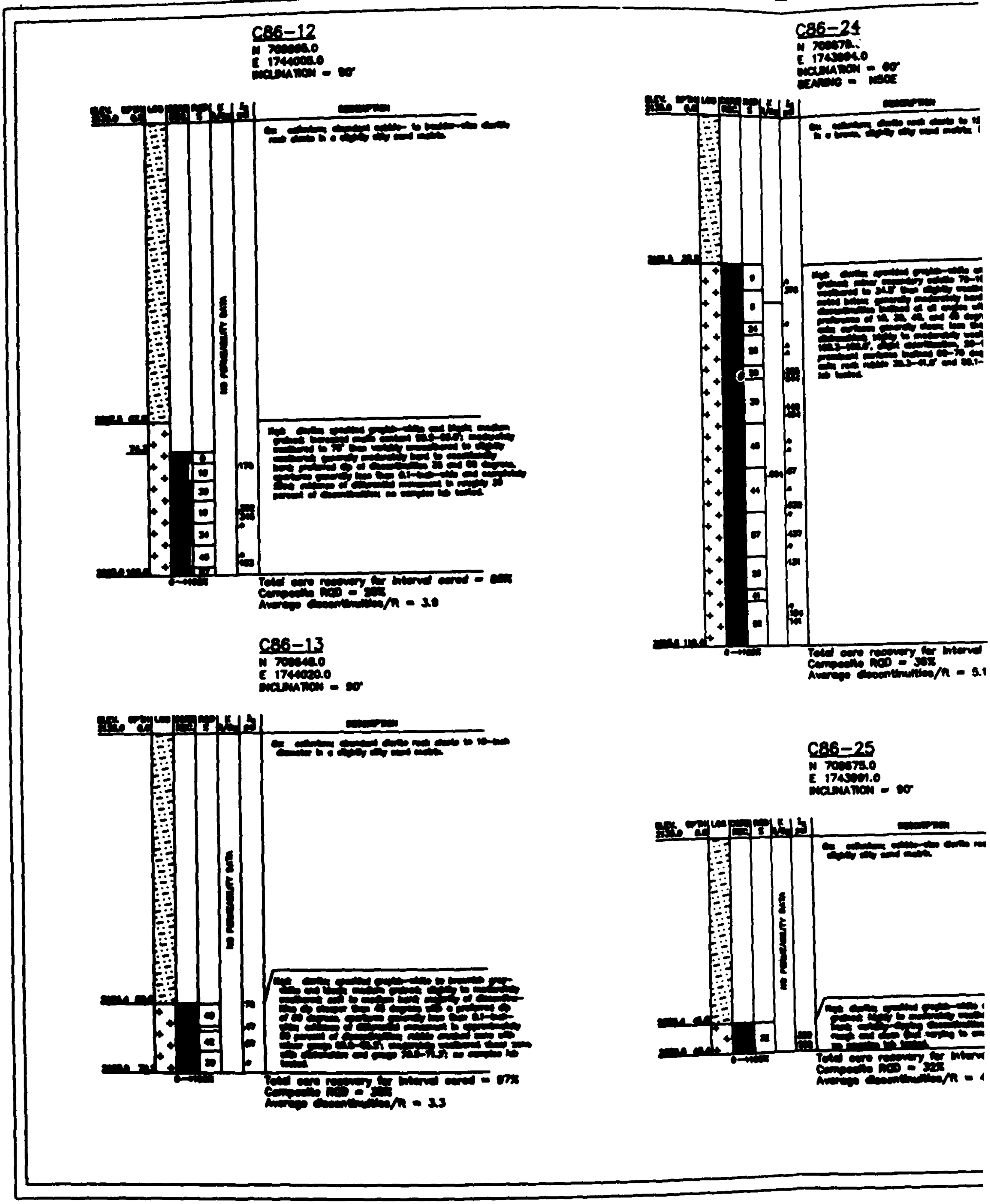
SEVEN OAKS DAM
OUTLET WORKS

Core Logs
Outlet Works, Inlet Portal

SUBMITTED BY: STEVEN L. STOCKTON, P.E. CHIEF, GEOTECHNICAL BRANCH

DISTRICT FILE NO. OAK-80-1/11

PLATE B-2


C86-12
E 1744005.0
INCLINATION = 90°
DESCRIPTION
NO PERMEABILITY DATA
Total core recovery for interval cored
Average discontinuities/ft = 3.9
C86-13
E 1744020.0
INCLINATION = 90°
DESCRIPTION
NO PERMEABILITY DATA
Total core recovery for interval cored = 97%
Average discontinuities/ft = 3.3
C86-24
INCLINATION = 60°
DESCRIPTION
Total core recovery for interval
Composite RQD = 38%
C86-25
N 708875.0
E 1743991.0
INCLINATION = 90°
DESCRIPTION
NO PERMEABILITY DATA
Total core recovery for interval
Composite RQD = 32%

C86-24

N 708678.[illegible]
E 1743864.0
INCLINATION = 60°
BEARING = N50E

DESCRIPTION
Qc colluvium; diorite rock clasts to 12-inch diameter in a brown, slightly silty sand matrix; firmer below 6'.
Kqd diorite; speckled grayish-white and black; medium grained; minor secondary calcite 70-105'; moderately weathered to 34.5' then slightly weathered except as noted below; generally moderately hard to very hard; discontinuities inclined at all angles with a slight preference of 10, 30, 45, and 75 degrees to the core axis; surfaces generally clean; less than 10 percent slickensided; highly to moderately weathered shear zone 103.3-105.6'; slight disintegration, 20-60 percent gouge, prominent surfaces inclined 60-70 degrees to the core axis; rock rubble 38.3-41.0' and 58.1-60.0'; no samples lab tested.

Total core recovery for interval cored = 98%
Composite RQD = 38%
Average discontinuities/ft = 5.1

C86-25

N 708675.0
E 1743861.0
INCLINATION = 90°

NO PERMEABILITY DATA

DESCRIPTION
Qc colluvium; cobble-size diorite rock clasts in a slightly silty sand matrix.
Kqd diorite; speckled grayish-white and black; medium grained; highly to moderately weathered; very soft to hard; variably-dipping discontinuities predominantly rough and clean (but varying to smooth and gouge-filled); no samples lab tested.

Total core recovery for interval cored = 100%
Composite RQD = 32%
Average discontinuities/ft = 4.4

LITHOLOGIC LEGEND

Overburden:

- Qal Alluvium
- Qc Colluvium
- Sand layer
- Boulder

Bedrock:

- Kqd Diorite
- Gn Gneiss

NOTES:

1. See plate B-1 for locations of core holes.

SYMBOL	DESCRIPTION	DATE	APPROVAL

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U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY:
CHECKED BY:

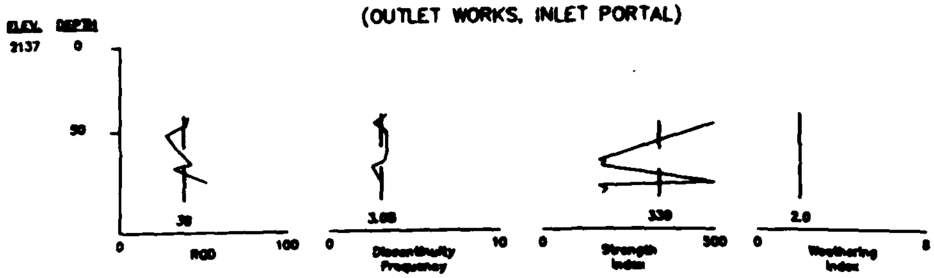
SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
OUTLET WORKS

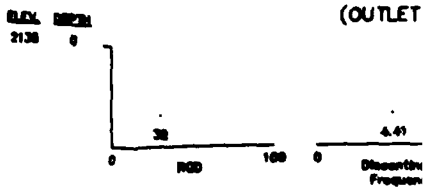
Core Logs
Outlet Works, Outlet Portal

SUBMITTED BY: STEVEN L. STOCKTON, P.E. [illegible]
DATE APPROVED:
SPEC. NO. DACW09-__ B-__
DISTRICT FILE NO. SAR-SO-I/12
PLATE B-3

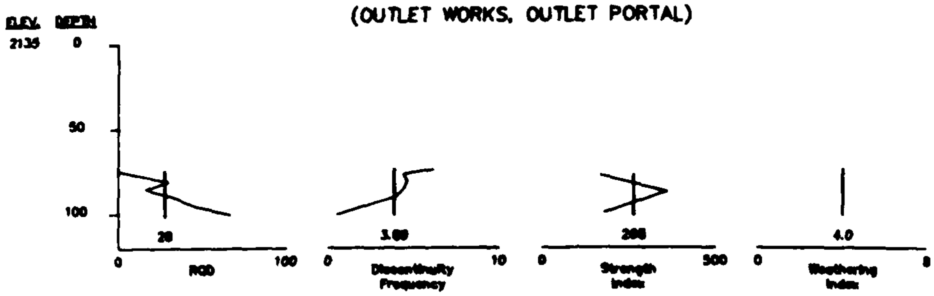
C86-07
(OUTLET WORKS, INLET PORTAL)



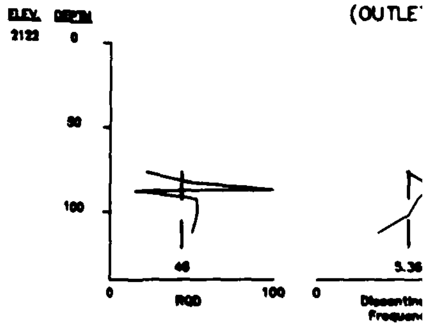
(OUTLET



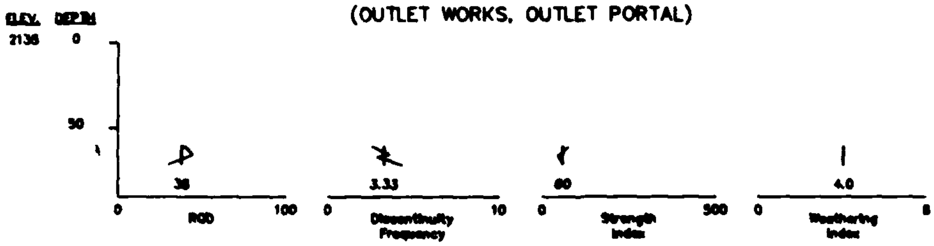
C86-12
(OUTLET WORKS, OUTLET PORTAL)



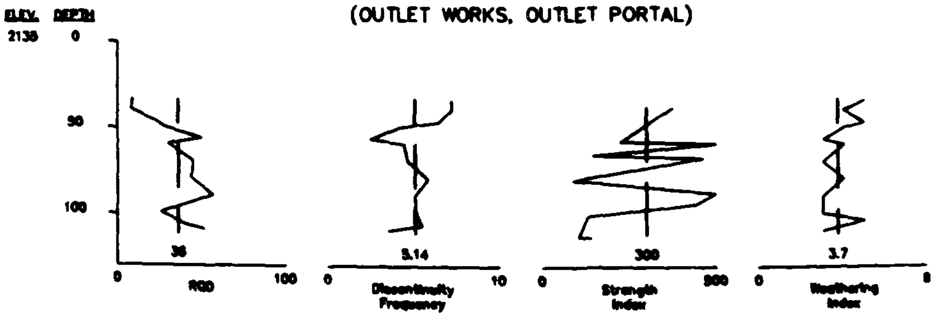
(OUTLE



C86-13
(OUTLET WORKS, OUTLET PORTAL)



C86-24
(OUTLET WORKS, OUTLET PORTAL)



C86-25
(OUTLET WORKS, OUTLET PORTAL)

C86-30
(OUTLET WORKS, INLET PORTAL)

EXPLANATION

RQD — The total length of intact core pieces greater than 4 inches in length divided by the length of each core run. The RQD (Rock Quality Designation) is expressed as a percentage.

Discontinuity Frequency — The number of incipient or broken discontinuities (fractures, joints, shears, etc.) per foot of recovered core. A value of 10.00 is arbitrarily considered to be the maximum.

Strength Index — The results of point load tests with size corrected to represent 50mm core. As a rough approximation, the unconfined compressive strength is about 20 times greater than the point load strength index. Results are expressed in psi. Strength index values over 500 psi (very hard rock) are plotted as 500.

Weathering Index — An arbitrary numerical scale of core sample weathering based upon field inspection using Bieniawski's (1979) classification method.

Value	Weathering Description
0	Unweathered
1	Unweathered to Slightly Weathered
2	Slightly Weathered
3	Slightly to Moderately Weathered
4	Moderately Weathered or Slightly to Highly Weathered
5	Moderately to Highly Weathered
6	Highly Weathered or Moderately to Completely Weathered
7	Highly to Completely Weathered
8	Completely Weathered

NOTES:

1. See plate 8-1 for locations of core holes.

SYMBOL	DESCRIPTION	DATE	APPROVAL

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LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY
DRAWN BY
CHECKED BY

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

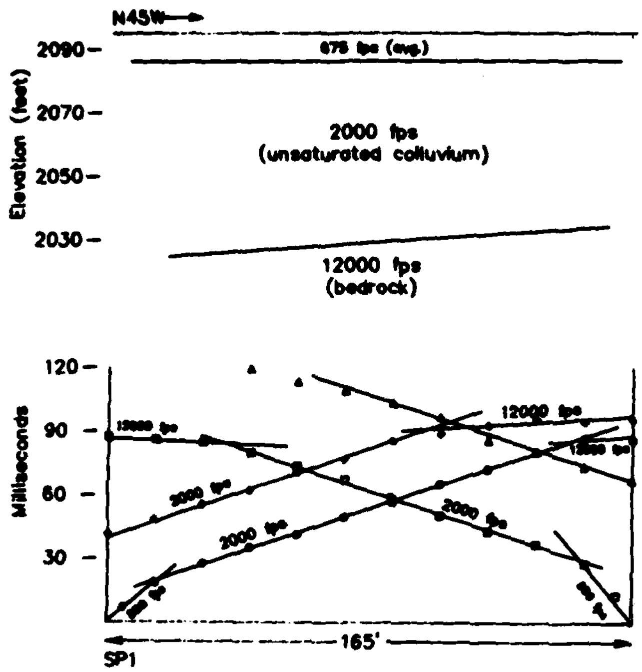
SEVEN OAKS DAM
OUTLET WORKS

Rock Quality Parameter Comparisons
Outlet Works

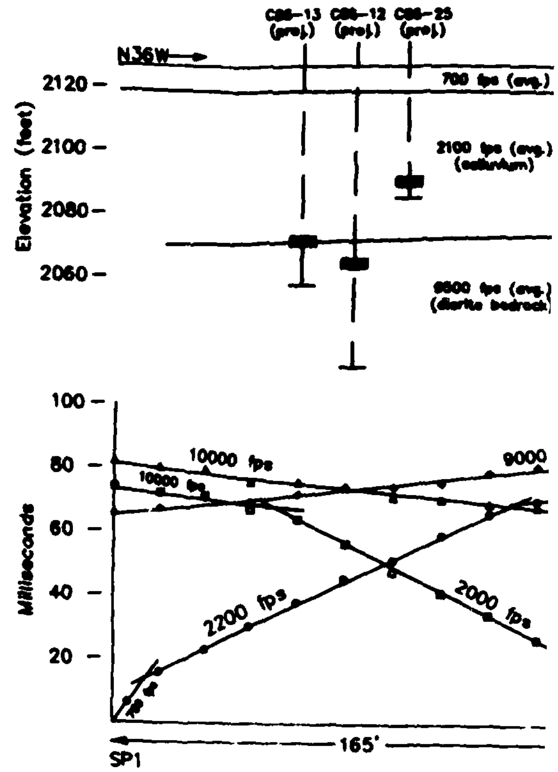
SUBMITTED BY
DATE APPROVED
STEVEN L. STOCKTON, P.E.
CHIEF, GEOTECHNICAL BRANCH

SPEC NO. DACW09- _ _ B _ _
DISTRICT FILE NO.
OAK-20-1/13

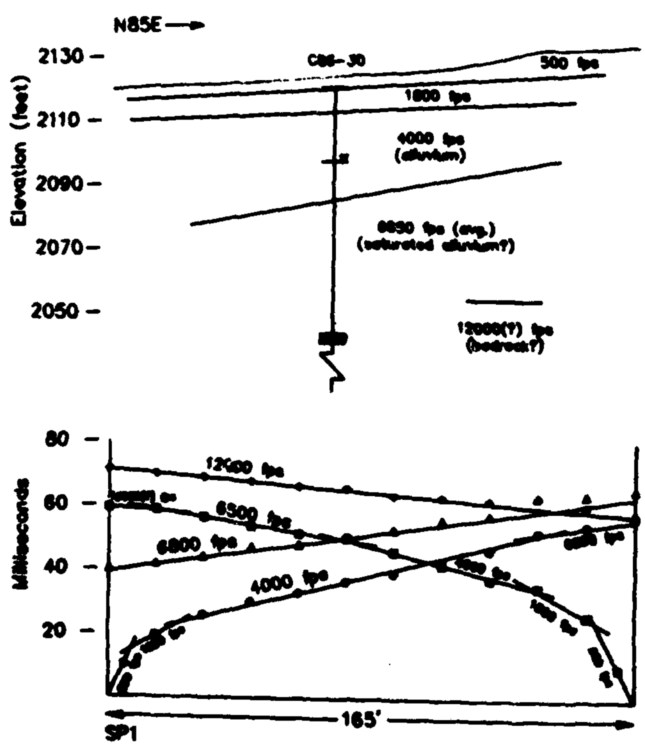
PLATE
8-4



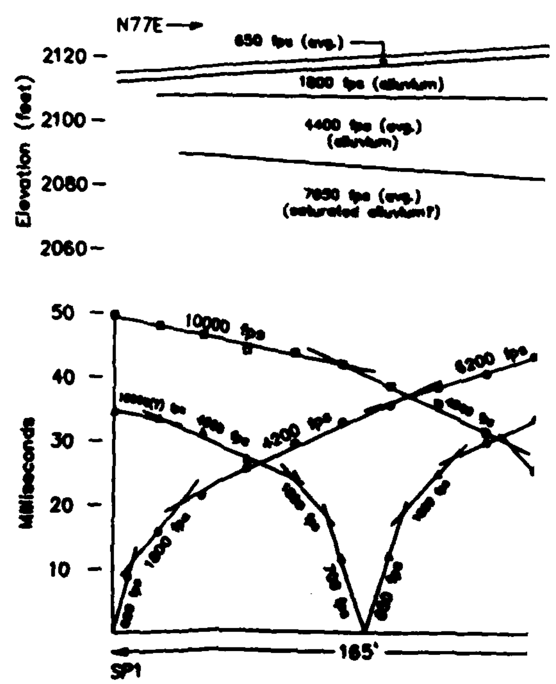
86-03



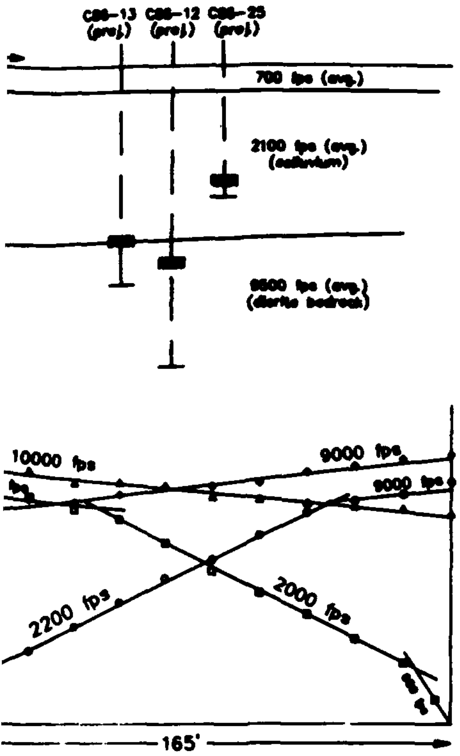
86-08



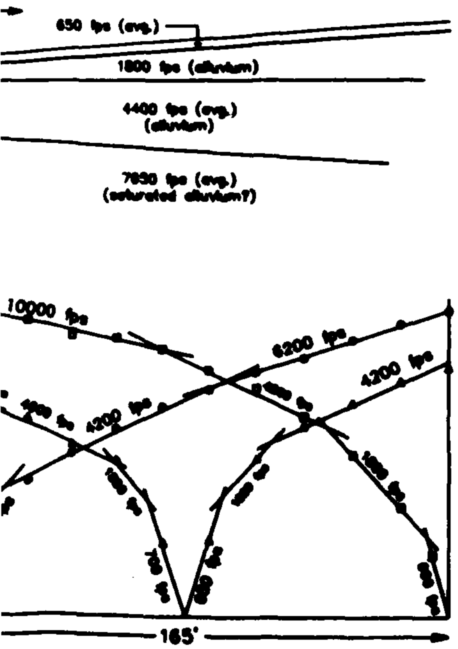
87-03



87-05



86-08



87-05

LEGEND:

- O Shot Point 1 (SP1)
- □ Shot Point 2 (SP2)
- Δ Shot Point 3 (SP3)
- ◇ Shot Point 4 (SP4)
- ⊕ Shot Point 5 (SP5)

NOTES:

1. See plate B-1 for locations of seismic lines.
2. Interface depths calculated using the critical distance method.
3. Seismic refraction lines 86-04 and 86-06 were conducted downstream at an alternative damsite and subsequently dropped from consideration. Results of these surveys are not pertinent to the selected damsite.

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U.S. ARMY ENGINEER DISTRICT PORTLAND CORPS OF ENGINEERS	U.S. ARMY ENGINEER DISTRICT LOS ANGELES CORPS OF ENGINEERS

DESIGNED BY:

DRAWN BY:

CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
OUTLET WORKS

Seismic Refraction Surveys
Traveltime Curves and Interpetation

SUBMITTED BY: STEVEN L. STOCKTON, P.E. CHIEF, GEOTECHNICAL BRANCH

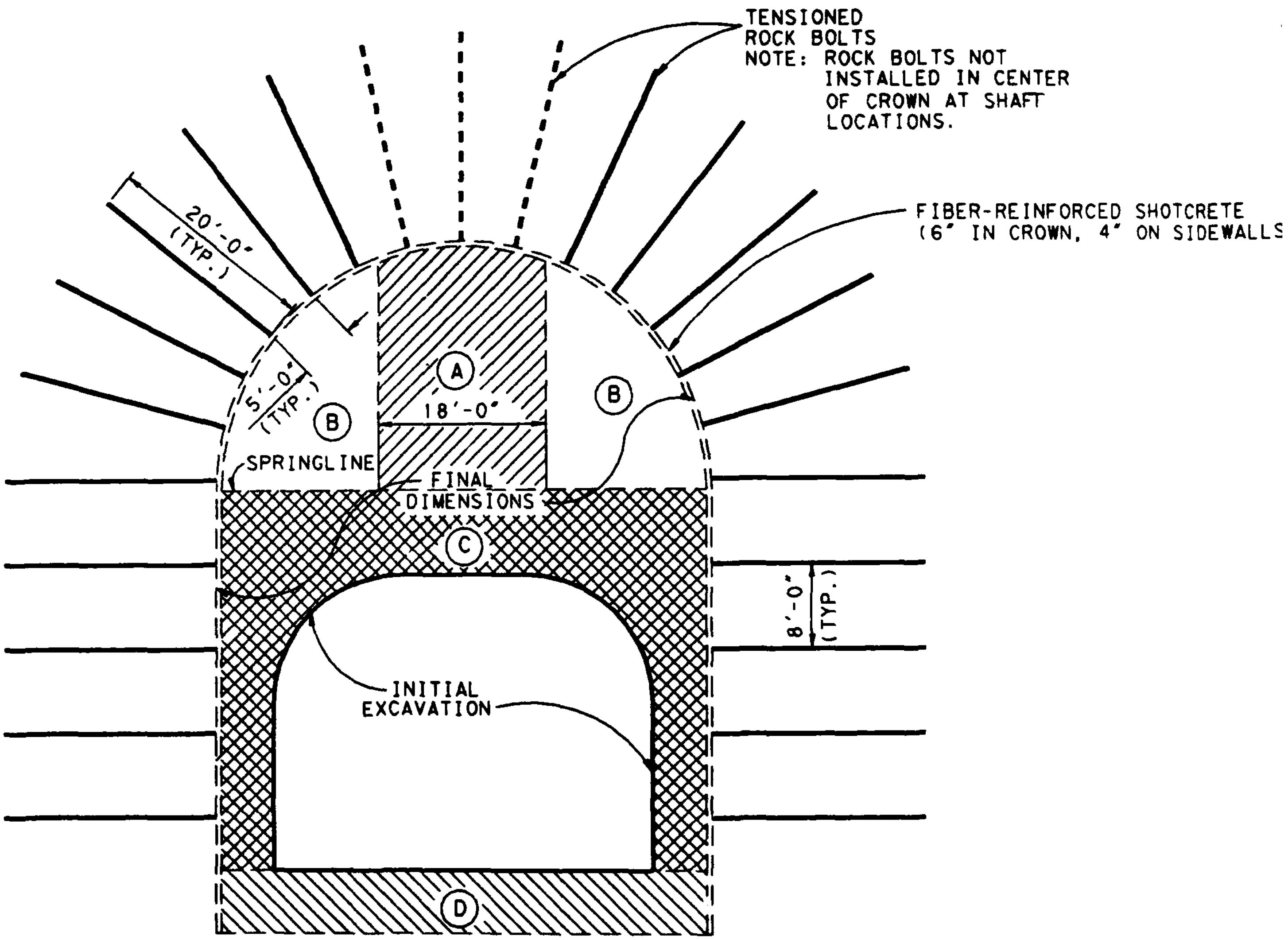
DATE APPROVED

SPEC. NO. DACW09-__ B __

DISTRICT FILE NO. OAK-20-1/14

PLATE B-5

GATE CHAMBER E
AND
SUPPORT DE



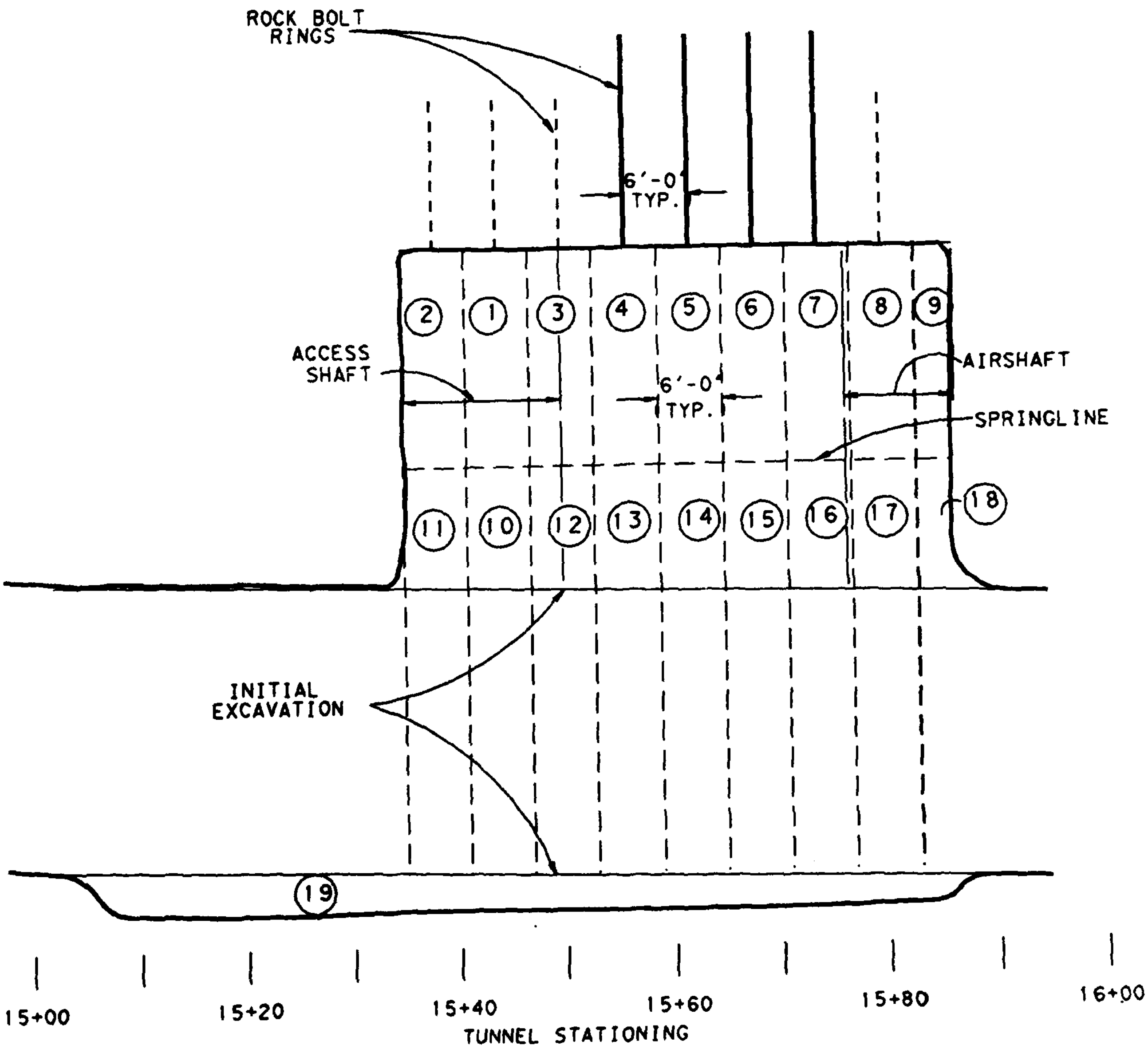
MID-TUNNEL CONTROL EXCAVATION
TYPICAL SECTION

LEGEND

(2)(B) EXCAVATION SEQUENCE

ER EXCAVATION
AND
RT DESIGN

OTCRETE
SIDEWALLS)



ENCE

MID-TUNNEL CONTROL EXCAVATION
℄ PROFILE

PLATE 8-6

SECTION 9

MECHANICAL DESIGN

9.1 General. This section describes the mechanical equipment and systems required for operation and presents basic data, design criteria, assumptions, and methods that will be used in the design of the mechanical features of the Seven Oaks Project. This equipment includes the RO and low flow slide gates and hydraulic operating equipment, RO and low flow gate removal procedures, forebay water level sensor, multiple level withdrawal system, maintenance bulkheads, RO and low flow conduit fill pipes, emergency generator, deckwash system, sanitary system, structural drains, compressed air system, and ventilation requirements. These systems are required for all three control options; however, they vary somewhat in size and location. An elevator for use solely in the mid-tunnel control option is also described.

9.2 Regulating Outlet and Low Flow Slide Gate System. The RO and low-flow gates are shown on plates 2-7, 2-8, 3-7, 3-8, and 4-8. The system will consist of four 5-foot-wide by 9-foot-high RO gates and two 2-foot-wide by 3 1/2-foot-high low flow withdrawal gates. The bonnets and frames for the gates were designed for an internal and external load equal to the maximum pool pressure without exceeding a basic stress of 16,200 pounds per square inch (psi), using ASTM A-36 steel. External tension anchors into the concrete will be used to reduce the span in the gate frames. The design heads were calculated to the gate centerline from maximum pool elevation. The seals will be an aluminum-bronze alloy on the leaf, and stainless steel (ASTM A-314-72 type 303) on the frames and upper sill, as recommended by "Sliding and Friction Tests of Bearing Materials," performed in July 1960 by the University of Idaho, Engineering Experiment Station, Moscow, Idaho, for the U.S. Army Corps of Engineers, Walla Walla District, Washington. The bottom seal will be a stainless steel-tipped gate to a babbitted, ASTM B23-83, Alloy 3, flat sill. Both the stainless steel and aluminum bronze are weldable and machinable; therefore, repairs and attachment can be made by welding. A starting coefficient of friction

of 0.6 was used for the design calculations. RO and low flow slide gates will be operated by hydraulic cylinders. Design of the cylinders, flanges, cylinder heads, and other appurtenant parts will be in compliance with Section 8 of the ASME Boiler and Pressure Vessel Code for Unfired Pressure Vessel. The gate stem will be made of solid stainless steel type 304. The gate leaf will be designed for the maximum design head at a maximum basic stress of 16,200 psi using ASTM A-36 steel. An indicator rod connected to the gate leaf, showing the gate opening, will be provided. The emergency gates will be provided with a gate latching mechanism in the top of each cylinder to hold the gate leaf in the fully open position.

9.3 Hydraulic Operating Equipment. Operation of the RO and low flow slide gates will be by hydraulic cylinders. Local and remote control pushbutton stations will be provided. Remote position indication will be provided in the control room. Two hydraulic systems will be provided: one system for the four regulating outlet slide gates and one system for the two low-flow slide gates. The hydraulic schematic for each of these systems is shown on plate 2-8. The hydraulic systems will operate at 2,000 psi. The test and maximum pressure for the system will be 3,000 psi. The pumps will be rated at 27 gallons per minute (gpm) at 2,000 psi for the RO system and 2.8 gpm at 2,000 psi for the low flow system. The capacities of each system, with one pump running, will provide a gate lift speed for one gate of approximately 1 foot per minute (fpm) with a pump speed not in excess of 1,800 revolutions per minute (rpm). Each hydraulic system will be valved in such a manner that one pump and motor can be removed while the other pump is operating. Additionally, the pumps will be valved so that both pumps can be operated simultaneously to double the flow and resultant gate operating speed. The motors, pumps, and pressure relief valves will be mounted directly to their respective oil storage reservoirs. The oil reservoir will be sized to contain the volume of oil used when closing all gates. The reservoir for the regulating outlet hydraulic system will be 120 gallons. The reservoir for the low-flow hydraulic system will be 20 gallons. The oil storage reservoirs will include suction line indicating filters, return line filters, an air vent, and sight gauge. The piping and fittings will be steel and rated at

3,000 psi. Solenoid-controlled four-way valves will control hydraulic fluid to raise or lower the RO service and low flow gates from the control room. Manually operated four-way valves will control raising and lowering the emergency gates. A pressure switch will disconnect the pump motor circuit when the gate leaf contacts the bottom seal or the bonnet cover.

9.4 Regulating Outlet Gate and Hydraulic Equipment Removal.

a. Upstream Control. An overhead bridge crane rated at 7.5 tons will be used in the gate room to disassemble and remove the slide gates as necessary for repair or maintenance. The following procedures will be followed:

(1) Lower the gate completely and dewater the upstream portion of the gate area.

(2) Drain the hydraulic fluid from the cylinder.

(3) Remove the cylinder and piston.

(4) Remove the bonnet cover.

(5) Attach a threaded cap with hook eye to the top of the stem.

(6) Disconnect the lower portion of the stem from the gate.

(7) Raise the stem out of the gate frame.

(8) Attach a threaded cap with hook eye to the bottom of the stem.

(9) Attach come-a-longs to the bottom hook eye.

(10) Disengage the crane end trucks (free wheel mode).

(11) Slowly lower stem to horizontal, adjusting come-a-longs as required.

(12) Place stem on dolly/cart system.

(13) Place "false stem" on the crane and insert into gate.

(14) Lift out gate.

The method for reinstalling the gate will be by reversing the above stated removal procedure. The crane will have a coverage envelope large enough to place different parts of a disassembled gate and operator in areas of the machine room to facilitate "on-site" repair, if desired. Since removal of cylinders and gate leaves is not a common occurrence (estimated to be at least 10-year intervals), transportation equipment such as dollys and powered carts should be rented on an as-needed basis, and not purchased outright.

b. Downstream Control. RO gates will be removed, when necessary, through hatches in the gate structure roof. A rented mobile crane can be used for this operation. Lifting eyes will be strategically located to assist in positioning smaller gate/valve components.

c. Mid-Tunnel Control. The method for removing and reinstalling the RO gates for the mid-tunnel control will be similar to the upstream control. Disassembled parts will be removed by crane up a hoisting shaft in the access tower.

9.5 Water Level Indicator. A pressure transducer based indicating device will be located in the intake structure to measure and record the reservoir water level. This device will be capable of transmitting data to the control room.

9.6 Multilevel Withdrawal System.

a. Wet Well Sluice Gate. The wet well gate will be a 5-foot by 7-foot sluice gate. This gate will be designed to withstand full pool (504 feet of head). The operator will be sized to allow operation of the gate under 165 feet of head. The operator will be a portable power unit.

b. Minimum Discharge Valves. Flow control valves will be provided in the minimum discharge pipe. The size and location of the valves will be as shown on plates 2-4, 3-6, and 4-5. Two valves will be provided: one knife gate valve used for throttling purposes, and one ball valve used strictly for stopping the flow to facilitate removal and repair of the knife gate valve. These valves will be sized to withstand full pool (545 feet of head) and the electrical actuators will be sized to operate at 250 feet of head. Air injection downstream of the knife gate valve will be investigated to reduce potential cavitation.

9.7 Maintenance Bulkheads. Maintenance bulkheads will be provided for the RO conduit, the minimum discharge conduit for mid-tunnel and downstream control, and the low flow bypass conduit for downstream control as shown on plates 2-4, 3-4, and 4-4. The maintenance bulkheads will be located in the intake tower. Each bulkhead will be designed to withstand full pool head of 504 feet. They will be raised and lowered with a portable crane from the access road on the intake tower. Wire rope pendants will be permanently attached to the bulkheads and will be securely tied off when the bulkheads are either in the raised or lowered position. Bulkhead guides will be required to ensure alignment of the bulkheads with the conduit openings. The bulkhead will only be installed or removed in low water periods, and will be stored in a raised position in the bulkhead slots when not in use.

9.8 Low Flow Conduit Fill Pipes. Provisions will be made to fill the RO conduit between the emergency slide gates and the upstream maintenance bulkhead to equalize the head on the bulkhead prior to removal. Control valves will be located in the gate room. A tee connection will be installed upstream of the pump in the deckwash supply line to supply the conduit fill line.

9.9 Emergency Generator Plant. A diesel engine-driven generator will provide emergency power to the RO gate operating equipment, gate room, ventilating equipment, and other loads as described in section 10, "Electrical Design." A fuel oil storage tank with a 7-day capacity will be installed outside the gate structure. A day tank (8-hour capacity)

will be mounted near the generator to provide gravity flow of fuel to the engine fuel pump to facilitate engine starting. The main fuel tank will be filled by commercial truck from the outside. The fuel tank will be vented to the outside of the structure.

9.10 Deckwash Water System. An electric motor operated centrifugal pump will supply raw water for washdown purposes. The pump will be located in the RO gate room and will draw water from the wet well. The pump will supply 50 gpm at 50 psi to strategically located hose stations. Strainers will be supplied at the intake of the pump. The pump will be controlled from manual start/stop push-button stations located at the pump.

9.11 Sanitary Facilities. An electric incinerating toilet will be provided at the downstream end of the RO tunnel. It will be exhausted to the outside of the structure. A waterless hand cleaner will be provided in the restroom. Sewage disposal sites will not be required. Sanitary facilities will be provided.

9.12 Structure Drains. The RO gate room will be sloped to floor trench drains around the perimeter of the room. The trench will drain into the RO exit channel.

9.13 Compressed Air. A single, electrically operated, 5 hp, portable compressor will be provided to supply compressed air at 100 psi. This compressor will be stored in the gate room when not in use.

9.14 Ventilation. Fresh air will be supplied to the RO gate room. A minimum of one air change per hour will be ducted to the gate room. Exhaust will be vented through louvers located in the service area.

9.15 Elevator. A vertical lift, traction-type elevator will be provided in the mid-tunnel control room to transport personnel from below the intake deck to the machinery room floor. The elevator will be powered from a 460 volt, three-phase, 15 horsepower, motor-generator set mounted in an elevator equipment room above the elevator. The travel speed will be 350 feet per minute. There will be one emergency landing at approximately the midpoint of travel. The elevator will have a 2,000-pound load capacity.

SECTION 10

ELECTRICAL DESIGN

10.1 General. The electrical design will consist of the project power supply, emergency power supply, project power distribution and protection, power and control for electro-mechanical equipment and loads, lighting, communications, security, grounding, and corrosion mitigation.

10.2 Project Power Supply.

a. Normal Power. Electrical power for the project will be supplied from a 3-phase distribution line extension to be built and owned by Southern California Edison (SCE) for this project. The substation will consist of a load-break fused disconnect switch, surge arresters, 3-phase transformer, and a steel outdoor pad-mounted enclosure. The secondary service will be 480V, 3-phase, delta. The transformer will be located near the downstream access for upstream and downstream control schemes. It will be located near the access bridge for the mid-tunnel scheme; requiring the longest line extension.

b. Emergency Power. Electrical power during emergencies (i.e., no utility power source) will be generated on-site by a diesel-engine-driven generator. The engine will be equipped with automatic start-stop controls to ensure power availability at all times. An automatic transfer switch will ensure continuity of 480V, 3-phase service. The set will generally follow the requirements of CEGS-16264. The generator will be located near the downstream access for upstream and downstream control schemes. It will be located at the top of the control structure for the mid-tunnel scheme.

10.3 Project Power Distribution System.

a. 480 Volts. The main project power will be distributed at 480V, 3-phase (see plate 2-9, 3-9, and 4-9). Power will be from the secondary side of the utility transformer and supply a power center, through an

automatic transfer switch. The power center will supply loads required in the access structure and also the motor control center (MCC) in the gate room. MCC, supplied by the most economic means consistent with seismic requirements, will power and control all loads in the gate room. The best method for supplying MCC with cables almost 2,000 feet long for upstream control and 500 feet long for mid-tunnel control will be subject to further study. Preliminary investigations indicate that cables in tray or on a messenger are favorable alternatives for upstream control. For mid-tunnel control, multiple vertical supports holding slack cables appears feasible. The panelboards and the motor control center will be per CE-2205.05.

b. 208Y/120 or 120/240 Volts. Power will be obtained from 480V step-down general purpose lighting transformers. These transformers will supply lighting panelboards that feed loads such as luminares, receptacles, incinerating toilet, and other small loads. Panelboards will be industrial-type utilizing industrial E-frame circuit breakers. Transformers will conform to applicable NEMA and ANSI standards.

10.4 Control Systems.

a. Machinery and Equipment. The RO and low flow gates will have open-stop-close controls with indicating lights and position indications. They will be capable of local manual or remote manual operation from the control room. The hydraulic pumping sets will have local-remote on-off control with automatic shutoff at a set high pressure. Controls will be designed for extremely safe and reliable operation. Controls for the ventilation system will be manual on-off controllable at the access structure. The deck wash pump will be operable locally and remotely from the access structure.

b. Reservoir Sensing. A pressure transducer with local and remote readouts will be utilized for sensing the pool level. A recorder will be provided, if deemed necessary.

10.5 Communications Systems.

a. Telephone. Telephone outlets will be provided in the gate room, access structure, elevator, and along the access tunnel. Service for an outside line will be obtained from a line extension built and owned by the local servicing utility, General Telephone. Line extension costs will be highest for mid-tunnel control scheme. Upstream and downstream control schemes will cost the same.

b. Intercom. An intercom system between the gate room and the control room will supplement the telephone system. The intercom will have its own battery back-up system.

10.6 Security.

a. Regulatory Requirements. Applicable portions of regulations will be incorporated into the design. The regulations referred to are: AR 190-51, "Security of Army Property at Unit and Installation Level," and DARCOM Supplement, FM 19-30, "Physical Security"; and TM 5-853-1, "Designing for Security."

b. Local Requirements. Special requirements will be added to mitigate vandalism indigenous to the area.

10.7 Lighting System.

a. Normal Operation and Maintenance Requirements. Illumination will be provided as necessary for safe operation and maintenance of the facilities. Luminaires will be of the energy efficient types, including high-pressure sodium and fluorescent light sources. Color rendition will be fully coordinated with the task involved.

b. Emergency. With the availability of the emergency generator, emergency type luminaires will be utilized where economic.

c. Security. Vandal resistant luminaires will be strategically placed for security illumination during darkness.

10.8 Raceways, Wire and Cable.

a. Raceways. Conduit will be used to the fullest extent. They will be of the rigid and intermediate types.

b. Wire and Cable. CE 1404.04 will be the guide for specifying wire and cable.

10.9 Grounding System. All metallic structures and equipment will be bonded together with bare copper cables. This network will be brought out to the pool where a ground mat will be buried.

10.10 Corrosion Mitigation. A water analysis will be made prior to the corrosion mitigation study. The analysis, study, and recommendations will be made during the FDM level.

10.11 Seismic Considerations. Electrical equipment with moving parts that depend on gravity for its performance, e.g. contactors and relays, have a seismic zone 4 (0.5 g) rating. Because of the 0.7 g criteria, spurious operation of outlet works gates, elevator, and pumps must be prevented. Leaving the power supply turned off, except during actual use, is the recommended method. Equipment such as fans, heaters, lighting, etc. may be left energized as spurious operation has no detrimental effects. However, design effort will be directed towards preventing spurious operation of water control equipment.

SECTION 11

OPERATIONS AND MAINTENANCE CONSIDERATIONS

11.1 General. Operations and maintenance activities for a project of this size will typically include a wide range of project operation functions, periodic and cyclic maintenance, and an extensive inspection and evaluation program. This section summarizes these activities and their associated costs.

11.2 Operations. Seven Oaks Dam will be operated strictly as a flood control project, personnel staffing will be based on on-site during operation. Operation (excluding maintenance) will consist of positioning the low-flow and RO slide gates as requried to reduce incoming flood flows to a non-flood flow downstream. Normally all flows will pass through either the low flow bypass or minimum discharge line. When inflows are high enough to cause the forebay elevation to rise to the spillway crest, all slide gates will be closed and all flow will then be discharged over the spillway.

a. Project Regulation.

(1) Main Regulating Outlet Gates. The minimum opening will be 9 inches for main RO gates. This limit should reduce undesirable vibrations and cavitation damage experienced at some existing projects which were operated at small gate openings under high head.

(2) Low Flow Bypass. The low flow bypass will be operated instead of the main RO gates, when main RO gate openings of less than 9 inches are required to control flow. See section 6 for discussion of gate openings. The minimum opening will be 6 inches for low flow bypass gates.

(3) Minimum Discharge Line. The minimum discharge line will be operated instead of the low flow bypass, when low flow bypass gate

openings of less than 6 inches are required to control releases. See Hydraulic Design, section 6.

b. General Operating Costs. Yearly operating costs for typical daily operations are assumed as follows:

Dam operator wages, benefits, etc.	$60,000
Vehicle and related misc.	$5,000
Temporary flood flow support	$30,000
Facility maintenance	
buildings, grounds, bridge and roads	$20,000
Total	$115,000

c. Inspection and Monitoring. There are several inspection programs which may include personnel other than project forces. These activities include but are not limited to the following:

(1) Periodic inspections.
(2) Evaluations.
(3) Bridge inspections.
(4) Reservoir monitoring.
(5) Surveys.
(6) Seismic service (USGS).

Annual costs for these activities are listed in table 11-1.

Table 11-1. *Annual Inspection/Monitoring Costs*

Item	Estimated Annual Cost
Periodic inspection	$15,000
Evaluations	8,000
Bridge inspection	500
Reservoir inspection	3,000
Survey monitoring	5,000
Seismic service (USGS)	1,000
Total	$32,500

11.3 Maintenance.

a. Debris.

(1) Debris Pool. A debris pool will be maintained at El. 2,200 for the initial project years. As the reservoir sediment level rises, the yearly debris pool level will rise also. The 100-year sediment level is predicted at El. 2,265, with a corresponding debris pool of El. 2,300. The debris pool is maintained during the wet portion of the year. During the dry season the pool is lowered in accordance with established reservoir regulation guidelines. Typically the pool will be lowered enough to create a "run of the river" condition. The rainy season will typically run between mid-November through the end of March. The end drawdown period may last from May in a dry year to August in a wet year. The pool is built up again with the "first heavy rain" which may occur as early as mid-October or as late as the end of December. After the debris pool elevation is reached, outflow will equal inflow up to 500 cfs. Releases up to 7,000 cfs may be discharged after the flood event at Prado Dam has passed. From 1 June to 1 July releases will equal inflow plus 10 cfs. From 1 July to 1 August releases will equal inflow plus 20 cfs. From 1 September to 31 October inflow will equal outflow for low flows, if a major event occurs, the debris pool will be established at that time.

(2) Debris Removal. Debris at Seven Oaks will consist of logs (from the upper basin), branches, brush, and grasses. Debris will be picked up once the pool recedes. A log boom may be required to help control the positioning of the debris. Contracted crews will pick up the debris once a year. The work will be performed during the dry season and could take as much as 30 days. The debris possibly could be burned on-site or will have to be removed for disposal at a landfill off-site. Heavy equipment may be required; dump trucks, dozers, and crane/loaders. Costs will range significantly, depending specifically with debris type, volume, and disposal requirements. Assuming 3 weeks, two crews, heavy equipment rental, and disposal off-site, $50,000 per year is used to estimate debris handling at Seven Oaks.

b. Concrete Repair. The intake structure and downstream channel structures will be exposed to flows that may erode concrete. Erosion may occur from abrasive sediments (mis-operation) or from high velocity cavitation at concrete discontinuities. Periodic inspections will identify these areas for future monitoring and/or repair.

c. Steel Maintenance - Downstream Control. The 11-foot main RO conduit will require repainting as conditions warrent. The upstream portion of the tunnel may be relatively moist, while the majority of the tunnel will be dry. An epoxy-polymide three-coat paint system (environmental zone 2A-1e) with commercial blasting is assumed at a 15-year cycle. The total exterior of the conduit has 75,000 square feet of surface, including stiffners and supports. The cost of this work is estimated at $364,000 or an approximate annualized cost of $13,400. The thickness of all conduits are designed for the life of the project and will be monitored for unacceptable abrasion levels. Bulkhead gates, guides, gratings, and other frabricated steel work will require yearly inspection and maintenance (estimated at $2,500 per year).

d. Mechanical.

(1) Regulating Outlet (RO) Gates. The maintenance of the RO gates will follow a consistent repair and inspection schedule. Quarterly, annual, and 5-year inspections are carried out by various operations and engineering personnel. Minor repairs are estimated to occur every 10 years: typically motor/pump failure, valve replacement, or gland replacement. Major repairs due to seal failure or gate tip failure would be expected every 30 years. These failures would require removal of the gate leaves.

(2) Other Mechanical Equipment. Other maintenance is considered normal for each mechanical system. The proposed mechanical maintenance items and costs are shown on tables 11-2.1, 11-2.2, and 11-2.3. Costs are annualized over 100 years at 8 7/8 percent interest.

e. Electrical. Most of the equipment will require scheduled preventative maintenance (PM). Maintenance will range from checking the equipment's operation to routine replacement of parts, e.g., from an "on" and "off" operation to the replacement of load contacts of contactors. The expected life of nearly all equipment is less than the 100 years of the structure, therefore, much of the equipment will be replaced from one to five times. Tables 11-3.1, 11-3.2, and 11-3.3 summarize the equipment, expected life, PM requirement, and replacements required for the three RO alternatives. The most difficult figure to develop will be "unscheduled" maintenance, i.e., equipment failure in a statistical form. An average from a similar project will be the best source. There is also a "reverse" bell-curve in action for each project, i.e., maintenance will be higher than normal when the equipment is new and near end-of-life.

Table 11-2.1. Annualized Mechanical Operation and Maintenance Costs Upstream Control

Item	Description	Annualized Operation and Maintenance Cost
RO gates, operators, hydraulics, etc.	5-foot by 9-foot regulating outlet gates, gate frames, cylinders, hydraulic operating machinery, etc. Four gates, two service, and two emergency. Design head - 504 feet	$5,600
Low flow slide gate	2-foot by 3-foot 6-inch low flow bypass including operator (two required). Design head - 504 feet	3,500
Multiple-level withdrawal gate	5-foot by 7-foot wet well sluice gate, manual operator	200
Emergency generator, tank, etc.	Provide emergency power for operating RO bulkhead, RO gates, lights, low flow bypass	1,000
Minimum discharge line - valves	Emergency ball valve and operating valve for 16-inch minimum discharge line	500
Floatwells	Forebay water level sensors	100
Water supply	Fill RO conduit between slide gates and bulkhead, deckwash system with pump	500
Hoists/cranes	Maintenance in gate room, equipment removal, one at 10 HP, two trolleys at 1 HP	1,000
HVAC	Intake structure, gate room, minimum one air change/hr, ductwork must go up access	1,000
Sanitary facilities	Electric toilet, waterless hand cleaner	500
Low flow maintenance bulkheads/gating	Upstream maintenance bulkheads for low flow and minimum discharge lines	400
Air compressor	Portable compressor for minor requirements	300
	TOTAL	$14,600

Table 11-2.2. Annualized Mechanical Operation and Maintenance Costs Mid-Tunnel Control

Item	Description	Annualized Operation and Maintenance Cost
RO gates, operators, hydraulics, etc.	5-foot by 9-foot regulating outlet gates, gate frames, cylinders, hydraulic operating machinery, etc. Four gates, two service, and two emergency. Design head - 512 feet	$5,600
Low flow slide gate	2-foot by 3-foot 6-inch low flow bypass including operator (two required). Design head - 512 feet	3,500
Multiple-level withdrawal gate	5-foot by 7-foot wet well sluice gate, manual operator	200
Emergency generator, tank, etc.	Provide emergency power for operating RO bulkhead, RO gates, lights, low flow bypass	1,000
Minimum discharge line - valves	Emergency ball valve and operating valve for 16-inch minimum discharge line	500
Floatwells	Forebay water level sensors	100
Water supply	Fill RO conduit between slide gates and bulkhead, deckwash system with pump	500
Hoists/cranes	Maintenance and equipment removal in gate room	1,000
Elevator	Access to gate room	3,000
HVAC	Gate room, minimum one air change/hr, ductwork must go up access	400
Sanitary facilities	Electric toilet, waterless hand cleaner	500
Low flow maintenance bulkheads/gating	Upstream maintenance bulkheads for low flow and minimum discharge lines	400
Air compressor	Portable compressor for minor requirements	300
	TOTAL	$17,000

Table 11-2.3. Annualized Mechanical Operation and Maintenance Costs Downstream Control

Item	Description	Annualized Operation and Maintenance Cost
RO gates, operators, hydraulics, etc.	5-foot by 9-foot regulating outlet gates, gate frames, cylinders, hydraulic operating machinery, etc. Four gates, two service, and two emergency. Design head - 545 feet	$5,600
Low flow slide gate	2-foot by 3-foot 6-inch low flow bypass including operator (two required). Design head - 545 feet	3,500
Multiple-level withdrawal gate	5-foot by 7-foot wet well sluice gate, manual operator	200
Emergency generator, tank, etc.	Provide emergency power for operating RO bulkhead, RC gates, lights, low flow bypass	1,000
Minimum discharge line - valves	Emergency ball valve and operating valve for 16-inch minimum discharge line	500
Floatwells	Forebay water level sensors	100
Water supply	Fill RO conduit between slide gates and bulkhead, deckwash system with pump	500
HVAC	Gate room, minimum one air change/hr	400
Sanitary facilities	Electric toilet, waterless hand cleaner	500
Low flow maintenance bulkheads/gating	Upstream maintenance bulkheads for low flow and minimum discharge lines	400
Air compressor	Portable compressor for minor requirements	300
	TOTAL	$13,000

Table 11-3.1. Annualized Electrical Maintenance and Replacement Costs Upstream Control

Item	Description	Cost
1. Automatic transfer switch	a. 600V, 3-Phase, 75A. b. 35-year life. c. PM required. d. Two replacements required.	 350 15
2. DQ1 Panelboard	a. 600V, 3-phase, 225A, 10 ckts w/auto split bus. b. 35-year life. c. PM required. d. Two replacements required.	 350 15
3. DQ2 Motor control center	a. 600V, 3-Phase, 600A, four stacks w/combo starters and CB compt. b. 35-year life (7-year life on contacts and coils). c. PM required. d. Two replacements required.	 350 250
4. Motors and generators	a. 460V, 3-Phase, TENV and DP. b. 20-year life (7-year life on bearings). c. PM required. d. Four replacements required.	 945 550
5. Transformers	a. 480-208Y/120V, 3-Phase. b. 50-year life. c. PM required. d. One replacement required.	 350 10
6. Lighting Panel-boards (2)	a. 208Y/120V, 3-Phase, 24 ckt. b. 35-year life. c. PM required. d. Two replacements required.	 705 10
7. Lighting Ballasts	a. Two-lamp, 118-120V. b. 10-year life. c. PM not required. d. Nine replacements required.	 60
8. Lighting lamps	a. 4-foot fluorescent. b. 10-year life (20,000-hour standard rating). c. PM not required. d. Nine replacements required.	 15
9. Tunnel Cables	a. 6 kft 250 MCM, 100 kft #14. b. 30-year life. c. PM not required. d. Three replacements required.	 425
10. Unscheduled	All equipment.	4,400
	TOTAL	$8,800

*PM - Preventative Maintenance

Table 11-3.2. Annualized Electrical Maintenance and Replacement Costs Mid-Tunnel Control

Item	Description	Cost
1. Automatic transfer switch	a. 600V, 3-Phase, 75A. b. 35-year life. c. PM required. d. Two replacements required.	 350 15
2. DQ1 Panelboard	a. 600V, 3-phase, 225A, 10 ckts w/auto split bus. b. 35-year life. c. PM required. d. Two replacements required.	 350 15
3. DQ2 Motor control center	a. 600V, 3-Phase, 600A, four stacks w/combo starters and CB compt. b. 35-year life (7-year life on contacts and coils). c. PM required. d. Two replacements required.	 350 250
4. Motors and generators	a. 460V, 3-Phase, TENV and DP. b. 20-year life (7-year life on bearings). c. PM required. d. Four replacements required.	 945 550
5. Transformers	a. 480-208Y/120V, 3-Phase. b. 50-year life. c. PM required. d. One replacement required.	 350 10
6. Lighting Panel-boards (2)	a. 208Y/120V, 3-Phase, 24 ckt. b. 35-year life. c. PM required. d. Two replacements required.	 705 10
7. Lighting Ballasts	a. Two-lamp, 118-120V. b. 10-year life. c. PM not required. d. Nine replacements required.	 60
8. Lighting lamps	a. 4-foot fluorescent. b. 10-year life (20,000-hour standard rating). c. PM not required. d. Nine replacements required.	 15
9. Tower Cables	a. 1.5 kft #2, 25 kft #14. b. 30-year life. c. PM not required. d. Three replacements required.	 80
10. Unscheduled	All equipment.	4,400
	TOTAL	$8,455

*PM - Preventative Maintenance

Table 11-3.3. Annualized Electrical Maintenance and Replacement Costs Downstream Control

Item	Description	Cost
1. DQ1 Motor control center	a. 600V, 3-Phase, 600A, four stacks with combination starters and ckt bkr compartments.	
	b. 35-year life (7-year life on contacts and coils).	
	c. PM required.	350
	d. Two replacements required.	250
2. Automatic transfer switch	a. 600V, 3-Phase, 75A.	
	b. 35-year life.	
	c. PM required.	350
	d. Two replacements required.	15
3. Motors and generators	a. 460V, 3-Phase, TENV and drip proof.	
	b. 20-year life (7-year life on bearings).	
	c. PM required.	935
	d. Four replacements required.	550
4. Transformers	a. 480-208Y/120V, 3-Phase, dry-type.	
	b. 50-year life.	
	c. PM required.	350
	d. One replacement required.	10
5. Lighting Panel-boards (2)	a. 208Y/120V, 3-Phase, 24 Ckts.	
	b. 35-year life.	
	c. PM required.	705
	d. Two replacements required.	10
6. Lighting Ballasts	a. Two-lamp, 118-120V.	
	b. 10-year life.	
	c. PM not required.	
	d. Nine replacements required.	60
7. Lighting lamps	a. 4-foot fluorescent.	
	b. 10-year life (20,000-hour standard rating).	
	c. PM not required.	
	d. Nine replacements required.	15
8. Unscheduled	All equipment.	4,400
	TOTAL	$8,000

*PM - Preventative Maintenance

11.4 <u>Summary of Operation and Maintenance Costs</u>. Total annualized operation and maintenance costs for the Seven Oaks outlet works are as follows:

Table 11-4. Summary of O&M Costs

Activity		Annual Cost
General Operations		$115,000
Inspection and Monitoring		$ 32,500
Debris Removal		$ 50,000
Mechanical		$ 15,000
Electrical		$ 9,000
Steel/concrete		$ 20,000
	Total (rounded off)	$242,000

EMERGENCY ACTION PLAN

US Army Corps
of Engineers
Los Angeles District

SANTA ANA RIVER BASIN

SANTA ANA RIVER FLOOD CONTROL PROJECT

SEVEN OAKS DAM

SAN BERNARDINO COUNTY, CALIFORNIA

EMERGENCY ACTION PLAN

DURING CONSTRUCTION

August 1988

SEVEN OAKS DAM
San Bernardino County, California

EMERGENCY ACTION PLAN
DURING CONSTRUCTION

SEVEN OAKS DAM
FLOOD EMERGENCY PLAN DURING CONSTRUCTION

CONTENTS

PART 1 GENERAL INFORMATION

PART II EMERGENCY ACTION AND NOTIFICATION SUBPLAN

Contents (Continued)

PART I

GENERAL INFORMATION

1. Introduction

a. Authorization

ER 1110-2-1150, Engineering After Feasibility Studies, 24 June 1985

b. Purpose and Scope

This plan addresses emergencies related to the release of large volumes of water during high water levels caused by large storms at various embankment construction stages. Such conditions would arise if the dam were overtopped and rapid erosion of the embankment occured.

The emergency subplan presented herein describes procedures and means for detecting emergencies, notifying pertinent parties and initiating required emergency operations and repairs at the project.

Notifications specified are those necessary for:

- Prompt evacuation of downstream residents.
- Ensuring safety.
- Vacating project areas where emergency operations may be conducted.
- Internal Corps of Engineers notification.
- Coordination with non-federal units of Government and other Federal agencies.

2. Description of Project Area

a. Location

The damsite is located about 1 mile upstream from the mouth of Santa Ana River Canyon and about 4 miles north-northeast from the town of Mentone, California. The reservoir (at full-pool conditions) would extend approximately 3 miles upstream in Santa Ana River Canyon and would also inundate portions of Government Canyon and Warm Springs Canyon. The damsite and reservoir are ~~entirely~~ within the boundaries of the San Bernardino National Forest in San Bernardino County. See Plate-1.

b. Drainage Area

Upper Santa Ana River canyon, draining approximately 177 square miles, excluding the 32 square miles tributary to Baldwin Lake, has its headwaters in the rugged San Bernardino Mountains. Elevations vary from 10,664 feet at Anderson Peak and 11,502 at San Gorgonio Peak to 2060 feet at the damsite, which is approximately 1 mile upstream from the canyon mouth. Bear Creek, the principal tributary within the canyon area, comprises 55 square miles.

c. Topography

The damsite and reservoir area is within the steep-walled Santa Ana River Canyon along the southern margin of the San Bernardino Mountains. The elevation of the canyon floor at the damsite is approximately 2060 feet (NGVD). The gradient of the canyon floor averages about 3 percent. Elevations of the ridgetops outlining the canyon range from about 3500 feet directly above the damsite on the left (east) abutment, to almost 4000 feet on the western flank of the canyon. The canyon walls on the west side are very steep just above the canyon floor and then flatten at higher elevations. The "ridge and swale" topography on the east side of the canyon has more uniform slopes. Several "hanging valleys", probably created by the rapid uplift of the San Bernardino Mountains and equally rapid down-cutting by the Santa Ana River, can be identified nearby the site.

d. Geology

Upper Santa Ana River Canyon is within the southern frontal margin of the San Bernardino Mountains, which represent the eastern part of the Transverse Ranges physiographic and tectonic province in Southern California. The Transverse Ranges province is an elongated geomorphic and structural unit that trends essentially east-west, and is made up of chains of mountain ranges and valleys extending from the Santa Barbara Channel Islands eastward through the San Bernardino Mountains. Its principal geomorphic and structural features lie across the grain of adjacent physiographic provinces, which are strongly influenced by the San Andreas fault system. Within, and bounding the Transverse Ranges province, are other major fault zones that have been active during the same span of geologic time that the San Andreas system has been active. Rock units within the province are represented by Precambrian plutonic and metamorphic types, and complex sections of Cretaceous and younger rocks.

The San Bernardino Mountains represent the eastern segment of the Transverse Ranges physiograpic and tectonic province. This range rises to altitudes of 5000 feet to 11,000+ feet. It is made up of steep-walled deep canyons, a subdued and markedly discontinous upland surface, and several peaks and ridges that include San Gorgonio Peak (altitude 11,485 feet), the highest point in Southern California. The steep, south-southwest-facing margin of the range follows the trace of the San Andreas fault; the north margin is represented by a north-facing erosional escarpment. To the west, the San Bernardino Mountains are separated from the San Gabriel Mountains by the San Andreas fault. Rock units exposed within the range consist of Precambrian plutonic and gneissic rocks, Paleozoic quartzites, limestone, dolomite and marble, Mesozic plutonic rocks, terrestrial clastic rocks and basalt of Tertiary age, and Quaternary alluvial deposits.

e. Seismicity

(1) Regional Tectonics

The present tectonic regime is usually modeled with Southern California attached to the Pacific plate and moving about N35W relative to the North American plate. The juncture between these two plates is the active San Andreas fault system and its majors splays. In Southern California, these include the Mill Creek and Mission Creek faults (North Branch of the San Andreas), Pinto Mountain fault, Banning fault, and the South Branch of the San Andreas fault. Fault movement on these structures is right-lateral with several zones displaying a strong compressional component directed northward. The San Bernardino Mountains have been described (Sadler, 1983) as a Quaternary transpressional uplift (a stress regime which results in both lateral slip and compressionnal deformation) associated with the adjacent San Andreas fault and steep reverse faults along the northern range front, as well as laterally extensive thrust faults internal to the range (north of the project site).

(2) San Andreas Fault System

Seismic activity along the San Andreas fault system suggests that faulting related to the zone of weakness extends more than 700 miles from the Gulf of California to the Pacific Ocean north of San Francisco. The seismic behavior of the San Andreas fault system varies markedly along its length. There appear to be three distinct modes of behavior: (1) moderately frequent large shocks in the Salton Trough and Cape Mendocino zones (as have occurred several times in the past 100 years), together with significant aseismic creep; (2) frequent small to moderate-sized shocks together with fault creep along the segment from Hollister to Parkfield and (3) infrequent great earthquakes (e.g. 1906 and 1857) along the northern (Santa Cruz to Cape Mendocino) and "big-bend" (Cholame to San Bernardino, including the damsite area) segments.

(3) Other Major Faults

Numerous other major named and unnamed faults exist within approximately a 50-mile radius from the Seven Oaks damsite. These include the Newport-Inglewood fault, which caused the 1933 Long Beach earthquake, the Whittier-Elsinore fault zone, and the San Jacinto fault zone. Although each of these faults are capable of generating an earthquake in the magnitude range of 7 to 7.5 their greater distance from the damsite and smaller magnitude potential leave the San Andreas fault zone as the causative fault for the design earthquake at the site.

f. Climate

Three types of storms produce precipitation in the Santa Ana River Basin: general winter storms, local storms, and general summer storms.

General winter storms usually occur from December through March. They originate over the Pacific Ocean as a result of the interaction between polar Pacific and tropical Pacific air masses and move eastward over the basin. These storms, which often last for several days, reflect orographic influences and are accompanied by widespread precipitation in the form of rain and, at higher elevations, some snow.

Local storms can occur at any time of the year, either during general storms or as isolated phenomena. Those occurring in the winter are generally associated with frontal systems. These storms cover comparatively small areas, but result in high-intensity precipitation for durations of up to 6 hours.

General summer storms in this area are usually associated with tropical cyclones and occur very infrequently. They are known to have occurred in the late summer early fall months, but have not resulted in any major floods during the period of record.

3. Description of Project Features

a. Main Embankment

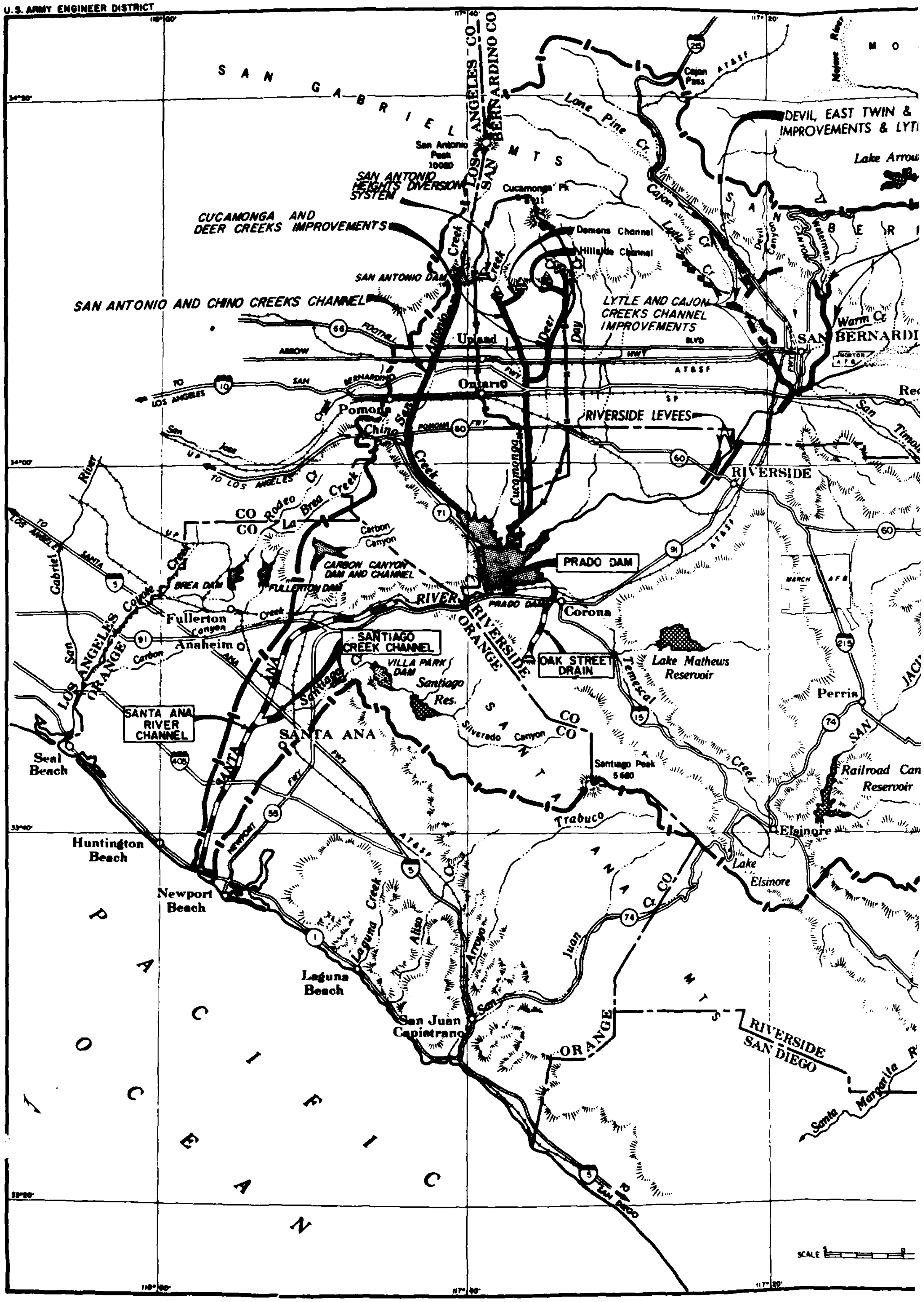
The top of the embankment would be at elevation 2610 feet above the National Geodetic Vertical Datum (NGVD) of 1929 and have a crest width of 40 feet. At the spillway crest elevation of 2580 feet above NGVD, the reservoir would contain approximately 145,000 acre-feet of gross storage. The embankment would be an arched earth-rockfill section with 1V on 2H outer slopes. The embankment would have five zones. See plates 2 and 3 for profile and cross sections.

b. Spillway

The unlined spillway would be 500 feet wide and excavated in bedrock on the left abutment and discharging into Deep Creek. The spillway will not be effective until the embankment reaches a height of 540 feet. Therefore, for the purpose of this emergency plan, the spillway is not considered operational.

c. Outlet Works

The recommended outlet works would consist of a high level intake tower, an 18-foot diameter regulating outlet diversion tunnel, an 11-foot diameter pressurized steel regulating outlet conduit, a downstream control and equipment structure located at the tunnel portal, and an outlet channel connecting the control structures to an energy dissipating plunge pool. During construction, the outlet tunnel will be ungated. Floodflows will therefore be uncontrolled. The maximum discharge of the 18-foot diameter diversion tunnel, before the 11-foot diameter steel conduit is installed, is approximately 27,500 cfs when the reservoir is at elevation 2,375 NGVD (SPF level of protection with an 18-foot diameter tunnel).


U.S. ARMY ENGINEER DISTRICT
SAN GABRIEL MTS
LOS ANGELES CO
SAN BERNARDINO CO
San Antonio Peak 10080
SAN ANTONIO HEIGHTS DIVERSION SYSTEM
CUCAMONGA AND DEER CREEKS IMPROVEMENTS
Demens Channel
Hillside Channel
SAN ANTONIO DAM
SAN ANTONIO AND CHINO CREEKS CHANNEL
LYTLE AND CAJON CREEKS CHANNEL IMPROVEMENTS
DEVIL, EAST TWIN &
IMPROVEMENTS & LYT
Lake Arrow
Lone Pine Cr.
Cajon Pass
Upland
Ontario
Pomona
Chino
SAN BERNARDI
RIVERSIDE LEVEES
RIVERSIDE
PRADO DAM
Corona
CARBON CANYON DAM AND CHANNEL
BREA DAM
FULLERTON DAM
Fullerton
Anaheim
SANTIAGO CREEK CHANNEL
VILLA PARK DAM
Santiago Res.
OAK STREET DRAIN
Lake Mathews Reservoir
SANTA ANA RIVER CHANNEL
SANTA ANA
Seal Beach
Huntington Beach
Newport Beach
Laguna Beach
San Juan Capistrano
Santiago Peak 5680
Perris
Railroad Canyon Reservoir
Elsinore
Lake Elsinore
SANTA ANA MTS
ORANGE
RIVERSIDE
SAN DIEGO
Santa Margarita
PACIFIC OCEAN
TO SAN DIEGO
SCALE

CORPS OF ENGINEERS

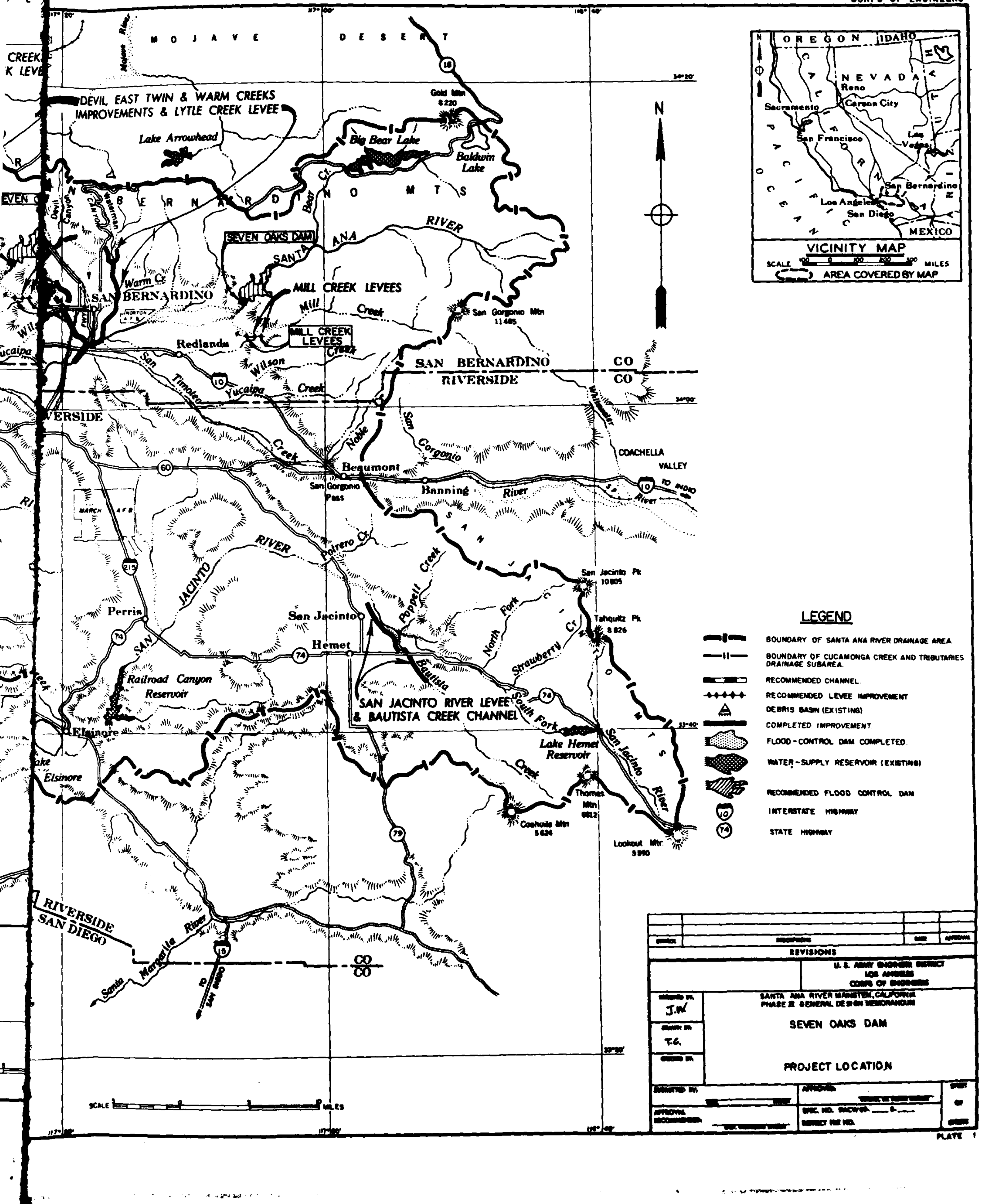
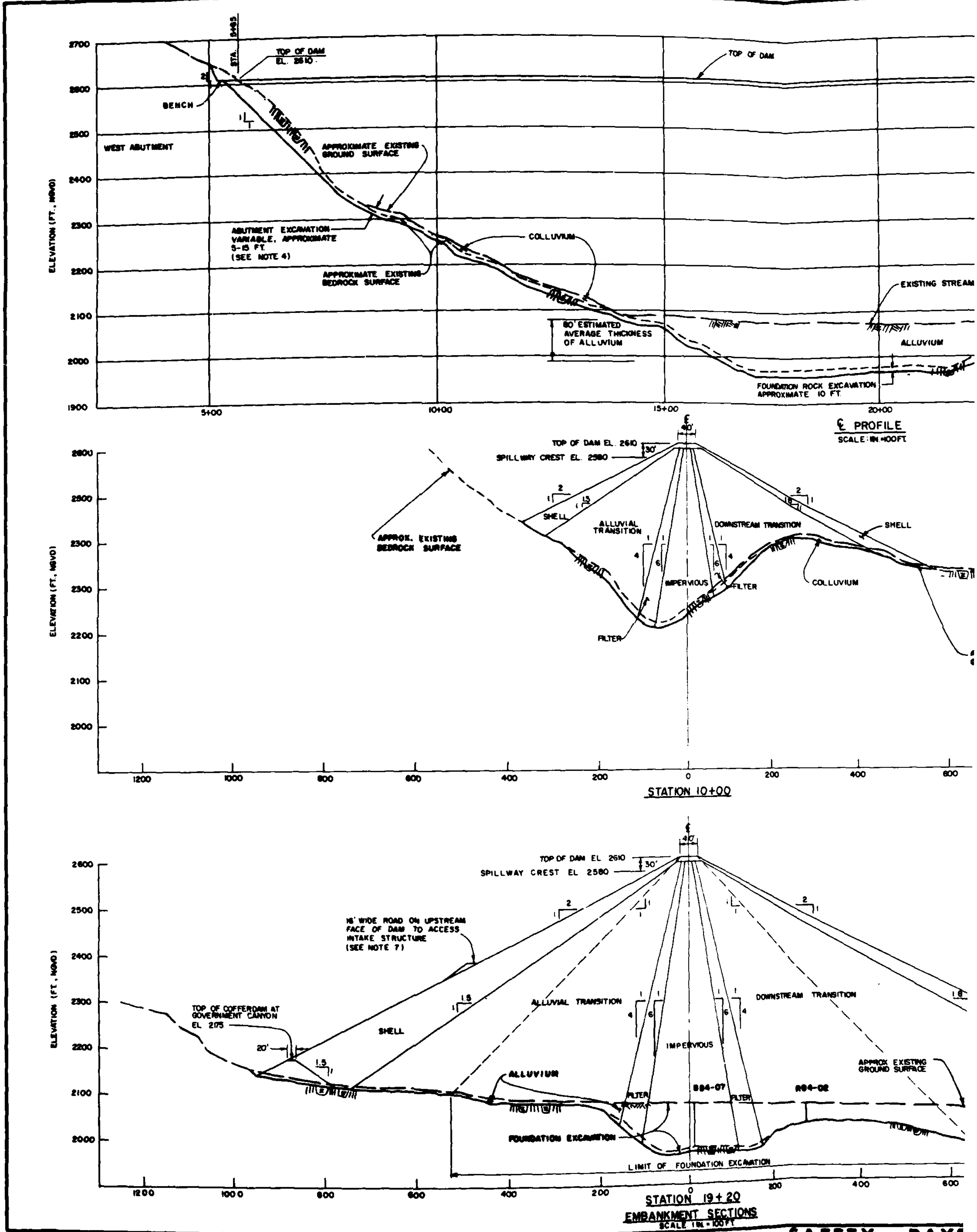
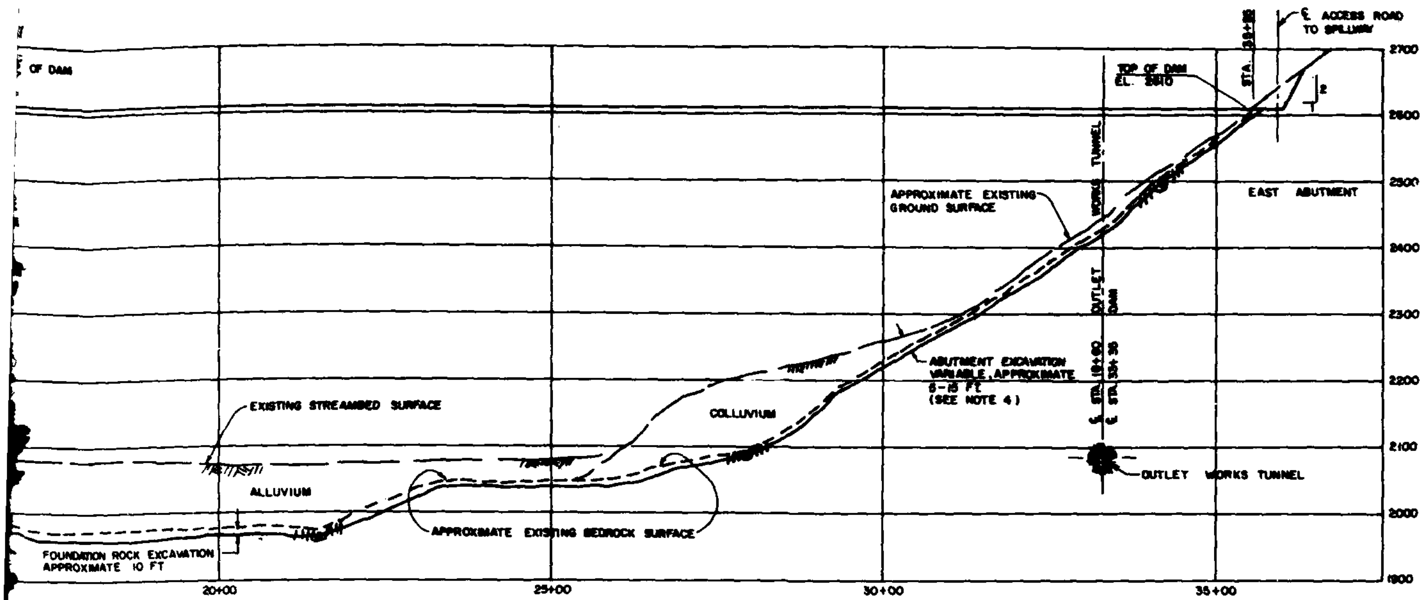


PLATE 1

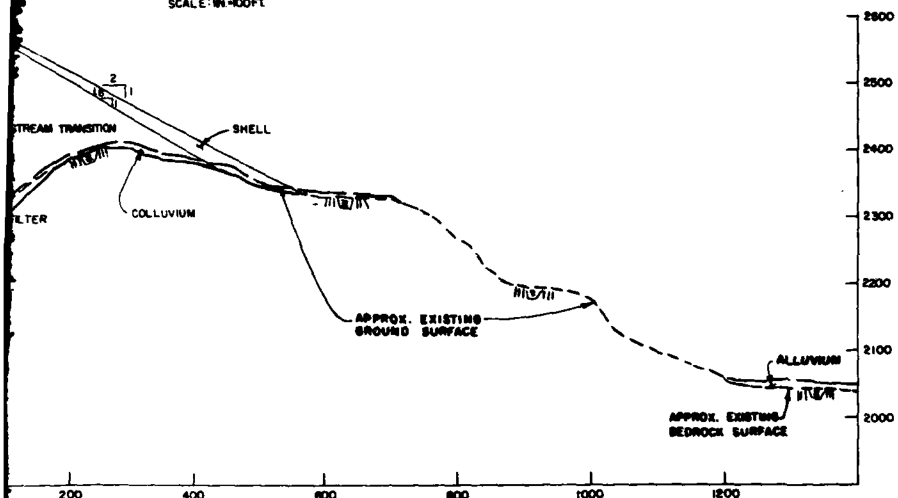
ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



LUE ENGINEERING PAYS

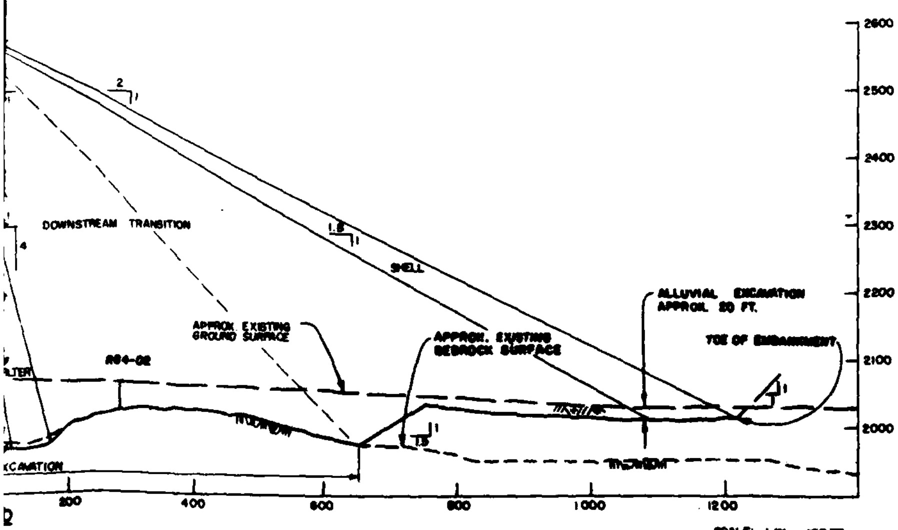


℄ PROFILE
SCALE: 1IN.=100FT.



NOTES:

1. ℄ PROFILE IS DRAWN LOOKING UPSTREAM.
2. EMBANKMENT SECTIONS ARE DRAWN LOOKING EAST.
3. BEDROCK SURFACE APPROXIMATED AND SMOOTHED.
4. FINAL CONFIGURATION AND DEPTH OF ABUTMENT EXCAVATION WILL BE DETERMINED IN DETAILED DESIGN.
5. B84-07, R84-02 DENOTE GEOTECHNICAL BECKER AND ROTARY DRILLINGS, YEAR DRILLED AND HOLE NUMBER.
6. SEE GEOTECHNICAL APPENDIX A, PLATE A-60, FOR EMBANKMENT MATERIAL GRADATIONS AND UPSTREAM COFFERDAM DETAIL.
7. SEE PLATE 21 FOR ACCESS ROAD DETAILS.



D
IONS

SCALE: 1 IN. = 100 FT.
100 0 100 200 300

DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SYMBOL	DESCRIPTION	DATE	APPROVAL

REVISIONS

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES
CORPS OF ENGINEERS

DESIGNED BY:
DRAWN BY: DE
CHECKED BY:

SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM

EMBANKMENT PROFILE AND SECTIONS

SUBMITTED BY:
DATE APPROVED:
SHEET
DISTRICT FILE NO.

PLATE 2

SAFETY PAYS

PART II

EMERGENCY ACTION AND NOTIFICATION SUBPLAN

EMERGENCY ACTION AND NOTIFICATION SUBPLAN

SEVEN OAKS DAM

1. Potential Causes of an Emergency

Potential causes of an emergency affecting the construction and safety of Seven Oaks Dam are described in the following subparagraphs.

a. Extreme storm

If an extreme storm were to occur over the watershed upstream of the project area it could result in large inflows to the reservoir and a rapid high reservoir level and waves on the reservoir surface.

b. Slope Failure

A sliding or sloughing of embankment fill could occur. A slope failure that extended to the top of the fill could effectively lower the crest of the dam. This could result in the sudden release of a large volume of water if the reservoir water surface overtops the dam and causes rapid erosion of the embankment.

2. Definitions

a. Uncontrollable Emergency. An uncontrollable emergency condition is one in which the occurrence of a significant hazard to life and/or property is certain to occur and no time is available to repair and/or modify operational procedures to prevent dam failure with subsequent uncontrolled water release.

b. Controllable Emergency. A controllable emergency condition is one not normally encountered in which the occurrence of a significant hazard to life and/or property is possible unless timely repairs and/or modifications are conducted to prevent dam failure and subsequent uncontrolled water release. Time must be available to eliminate the condition in order for the condition to be declared controllable.

c. Post-Earthquake Condition. If an earthquake occurs or one has been reported to have occurred with a Richter magnitude of 4.5 or greater within a 3-mile radius, 5.0 or greater within 30 miles, 6.0 or greater within a 100-mile radius from the dam a post-earthquake condition exists and a project inspection is required.

d. Extreme Inflow or Initial Filling Condition

e. Security Alert. A security alert relates to incidents at a project which could threaten the safety of the project.

f. Project Engineer. The project engineer is defined as the Corps of Engineers' on-site superintendent in charge of the project construction or his designated alternate.

g. Special Dam Inspection Team. The Special Dam Inspection Team (SDIT) consists of qualified District engineering personnel who perform onsite inspection(s) during declared extreme inflow, post earthquake, and security alert conditions so potential emergency situations will be detected in timely manner.

h. District Emergency Response Team. The District Emergency Response Team (DERT) consists of selected personnel who are to assist the chief of the Engineering Division in responding to an emergency condition at the project.

i. Reservoir Operations Center. The Reservoir Operations Center (ROC) is the command center (usually at the downtown District Office) from which water control decisions are determined. The required reservoir operations are coordinated through the ROC either by radio or telephone. The ROC is manned by the staff of the Reservoir Regulation Section. The number of staff at any given time is variable, depending upon the time of the day and the number and type of regulation decisions to be made. The ROC engineer could be represented by any of the Reservoir Regulation Section personnel on duty at that time.

3. Basis for Activation

This subplan will be activated upon occurence of an extreme inflow condition, initial filling condition, post earthquake condition or security alert.

4. Extreme Inflow Condition

a. Initial Filling. Based upon rainfall projections, the on duty ROC engineer will alert the Project Engineer of an incoming storm. The on duty ROC engineer will:

(1) Inform the Project Engineer of the expected rise in the reservoir.

(2) Provide updates based on new rainfall projections and reservoir level data supplied by the Project Engineer.

(3) Inform the Chief of the Water Management Branch at SPD of the incoming storm.

(4) Alert the SDIT using the list provided in Appendix C.

The Project Engineer will:

(1) Alert the DERT using Appendix B.

(2) Inform the on duty ROC engineer of changes in the reservoir level

An "Upstream Reservoir Inundation Map" that delineates the reservoir area that will be inundated at various reservoir elevations is included in Appendix I for reference.

b. Embankment Overtopping: If the ROC inflow evaluation indicates embankment overtopping will occur, the on duty ROC will immediately use the Emergency Notification List, Appendix A, to notify the authorities to evacuate people from the area downstream of the dam as shown on the downstream flood inundation maps included in Appendix H. These maps present inundation data for failure at various dam heights and are keyed to the time of uncontrolled water release.

The on duty ROC engineer will:

(1) Inform authorities of the estimated time when the initial overtopping will occur and the height of the dam.

(2) Contact the SDIT at the project, alerting them to the imminent overtopping of the dam.

c. Special Dam Inspection Team. When activated by the ROC, members of the SDIT or alternates (to be activated if members cannot be contacted) will report as soon as possible to Seven Oaks Dam. The mission of the activated SDIT under extreme inflow conditions is to perform continuous inspection of the project so an ongoing or potential emergency condition will be detected. Appendix C contains a checklist to be used during the inspection. While on duty the SDIT will maintain radio contact with the project engineer, ROC, and Chief, Emergency Management Branch. The chief of the SDIT is authorized to declare an uncontrollable or controllable emergency condition should an encountered situation(s) justify. First member to arrive at project will act as SDIT chief until designated SDIT chief arrives.

(1) Uncontrollable Emergency. Upon declaration of an uncontrollable emergency condition, the chief of the SDIT will immediately:

(a) Notify Downstream Interests. Use the Emergency Notification List, Appendix A, to alert downstream authorities of the need to evacuate people from the downstream floodplain. Since these notifications would occur during emergencies, telephones may be out. The Corps could provide the county water districts with Corps radios, so they could be hooked up to the Corps' emergency operations network. Report the type and nature of the problem and the estimated time when the initial uncontrolled water will be or has been released. Project downstream flood inundation maps that present inundation data, keyed to the time of uncontrolled water release, are included in Appendix H as a reference.

(b) Notify District Emergency Response Team. Alert one member of the DERT by using the District Notification List, Appendix B.

(2) Controllable Emergency. Upon the declaration of a controllable emergency, the SDIT chief will immediately use the District Notification List, Appendix B, to notify one member of the DERT. Suggestions for Corrective action for erosion on the dam is given in Appendix F.

(a) Information to Report. Report the condition, actions, and other information as indicated below.

1 Description of Condition:

- Time and location of incident.
- Nature and severity of problem(s).
- Present status of problem(s).
- Current and predicted reservoir condition(s), including water elevation, inflow, and discharge.
- Current and forecasted weather conditions.

2 Suggested Action(s):

- Type of corrective actions.
- Estimated time to complete corrective actions.
- Outlook for success.
- Assistance required/being furnished.
- Potential complications.
- Recommended downstream evacuation.

3 Other:

- Staff at damsite.
- Visitors at project.
- Road conditions.

5. Post Earthquake Condition.

It is the District's policy that Seven Oaks Dam be inspected immediately following a potentially damaging earthquake to ensure that the project's structural stability, safety, and operational adequacy is continued after the seismic event. The goal is to promptly detect conditions of significant distress, enable the conduct of timely restoration and remedial measures, and/or initiate notification procedures to provide maximum protection to downstream life and property.

a. District Notification. Under written agreement with USGS Earthquake Information Center (EIC), located in Denver, Colorado, the District is notified whenever a potentially damaging earthquake occurs within the jurisdiction of the District. The EIC will contact the District in about 1 hour after the seismic event occurs. The EIC will provide earthquake time of occurrence, magnitude (Richter sale), location (epicenter latitude and longitude), nearest community, and preliminary damage assessment. If the

contact is to be made during normal business hours, the EIC will contact the chief of geology. If however, the contact is to be made during off duty hours, the EIC will telephone one member of the DERT.

b. District Response. Upon receiving a telephone earthquake alert from the EIC, the chief of geology or the contacted member of the DERT is responsible for evaluating the earthquake information to determine the earthquake's potential for damage and, if required, initiating the District's response. To assist in the earthquake evaluation, Appendix D contains a form to record the earthquake alert data and maps to plot the epicenter so the distance between it and the dam can be approximately measured. An earthquake has the potential for damage, if it is 4.5 or greater magnitude (Richter scale) and the epicenter is within 3 miles of the dam, 5.0 or greater within 30 miles, 6.0 or greater within 50 miles, 7.0 or greater within 70 miles, or 8.0 or greater within 100 miles. If the earthquake evaluation indicates the seismic event has the potential to damage the project, then the chief of geology, or the contacted DERT member, will declare a post earthquake condition, immediately activate the SDIT using alert list in Appendix C for onsite inspection, and notify the other DERT members using Appendix B.

Severe earthquakes may disrupt communications, electric power, and transportation, and thereby prevent both notification by the EIC and formal mobilization of the DERT and SDIT. In the event that telephone service is disrupted and information from other sources and/or personal observations indicates an earthquake of Richter magnitude 7.5 or greater has occurred in or near the Los Angeles area, members of the DERT and SDIT will assume a post earthquake condition exists and mobilize without formal notification. DERT members will report to the District office. SDIT members will report directly to Seven Oaks Dam if during the rainy season of 1 October to 1 April and the District office at other times. Additional information regarding major earthquake procedures is presented in the "District Earthquake Response Plan" that was published in FY 1986.

c. Special Dam Inspection Team. Upon being activated under a post earthquake condition, members of the SDIT will report, as soon as possible, to Seven Oaks Dam. First member to arrive at project wil act as SDIT chief until designated SDIT chief arrives and will assure the SDIT follows this procedure:

(1) Visual Inspection. Immediately upon arriving at project conduct a general overall visual inspection of the project.

(2) Uncontrollable Emergency. If the visual inspection reveals that dam failure is imminent (uncontrollable emergency), initiate population notification/evacuation by immediatley contacting downstream authorities using Appendix A. Then alert District through use of the District Notification List, Appendix B.

(3) Controllable Emergency. If the results of the visual inspection reveal that damage and/or a changed condition(s) has occurred, then:

(a) Monitor and record the nature, location, extent of damage or changed condition(s) and other pertinent data; evaluate failure potential.

(b) Contact DERT (using Appendix B); it is vital that the report with damage description and hazard evaluation be made promptly and accurately.

(c) Continue to monitor damage and/or changed conditions and periodically report results to DERT.

(d) Thoroughly inspect project using the Post Earthquake Inspection Checklist, Appendix D.

(4) Unchanged Condition. If the results of the visual inspection do not reveal damage and/or changed conditions, then thoroughly inspect the project using the Post Earthquake Inspection Checklist, Appendix D.

(a) Contact the DERT (using the District Notification List, Appendix B) and report the results of checklist inspection. It is important that a "no damage" report be made.

(b) As damage and/or changed conditions to the project may not be readily apparent following an earthquake, inspect and monitor the project visually again at least 48 hours after the seismic event or as instructed by the DERT.

(c) First member to arrive at project will act as SDIT chief until designated SDIT chief arrives and will assure the SDIT follows this procedure:

6. Assistance and Resources

Equipment available at the District base yard for use in emergency situations and contractors available to provide equipment, labor, and materials are listed in Appendix E.

7. District Emergency Response Team.

The DERT (Team) is to assist the Chief, Engineering Division, in responding to an emergency condition at a District project. The Chief, Engineering Division, will direct the District's response efforts. In his absence, the Chief, Construction-Operations Division, will assume command. The Team consists of Chiefs of Operations, Geotechnical, Hydrology and Hydraulics, and Design Branch; Dam Safety Coordinator; and Chief, Emergency Management Branch (Team chief). Each Team member, with the exception of the Team chief, has an assigned alternate that will be a substitute should the Team member not be available.

a. Member Responsibilities During Team Activation. The Team will provide assistance to the Chief, Engineering Division, in analyzing and evaluating the situation(s), selecting and ordering key personnel to the project to provide onsite assistance, directing office staff, and developing an engineering solution(s). Dam Safety Coordinator will provide assistance to the division and branch chiefs as required. Chief, Emergency Management Branch, will alert pertinent members of the Los Angeles District staff, such as PAO, to assist in obtaining emergency construction equipment, personnel, and materials, and should it be necessary, be general liaisons between the Corps and other Federal and non-Federal agencies.

Upon the declaration of an emergency condition, post earthquake condition, or a security alert at a District dam, the Team will be alerted by one Team member or alternate being contacted. The contacted member or alternate is responsible for notifying the other Team members and/or pertinent District personnel as required, using District Notification List, Appendix B. After notification, the Team members and all contacted District personnel, if so ordered, will report as soon as possible to the office of the Chief, Engineering Division. Telephone/radio contact will be made between the Chief, Engineering Division, office and the project and the line kept open to facilitate communications.

b. Member Responsibilities - General. The Team may have to respond to a problem during on and/or off duty hours; consequently it is incumbent upon each Team member to keep the Chief, Emergency Management Branch, informed as to any changes in alternate work or home telephone numbers and to assure that the member and alternate maintain good familiarity with this flood emergency plan.

The Dam Safety Coordinator (DSC) will incorporate this subplan into the District's Dam Safety Training, and conduct annual exercises so all pertinent office and project personnel maintain familiarity with emergency action and notification procedures.

The Chief, Emergency Management Branch, is responsible for periodic updating of names and telephone numbers listed in this flood emergency plan and for coordinating procedures for notification of emergencies, with upstream and downstream authorities. Phone numbers will be updated semi-annually. To facilitate emergency communications, the Chief, Emergency Management Branch, will coordinate the purchase and maintenance of telephone beepers and additional radios. The beepers will be issued to the project engineer and ROC personnel. Base radios will be installed in the Reservoir Regulation Unit office and in the Emergency Management Branch office, and portable radios stored at the project for the use of the Special Dam Inspection Team. The Chief, Operations Branch, will conduct regular tests to verify beepers and radios are in good working order. The Chief, Emergency Management Branch, will obtain keys to pertinent District office electrical panels so office lights can be turned on, as required, by District emergency response personnel. Key sets will be provided to the Chiefs, Engineering and Construction-Operations Divisions, Dam Safety Coordinator, and Chief, ROC. A set will also be stored in the Emergency Management office.

8. Inquiries.

Project and District personnel should refer all inquiries from the news media and general public regarding an "emergency condition" to the District PAO (213) 894-5320. Examples of press releases for uncontrollable and controllable emergency conditions are presented in Appendix G for information only.

APPENDIX A

APPENDIX A
EMERGENCY NOTIFICATION LIST
SEVEN OAKS DAM

The parties listed below are to be notified immediately upon declaration of an uncontrollable emergency. Notification should include: (a) description of the type and extent of emergency that exists or is impending; (b) advice to evacuate people from flood plains; and (c) information on the time release of hazardous amounts of water began or is estimated to begin. Inundation maps indicating the travel time of a flood wave to points downstream from the dam are included in Appendix H.

a. San Bernardino County
   Communications Center

b. San Bernardino
   County Sheriff

c. Orange County Sheriff
   Communications Center-

d. Riverside Co.
   Disaster Preparedness

e. California Office of Emergency
   Services, Sacramento

Upon completing the above notifications, contact the District Emergency Response Team (Appendix B).

APPENDIX B

APPENDIX B
DISTRICT NOTIFICATION LIST
SEVEN OAKS DAM

The following list is to be used to alert the District Emergency Response Team upon declaration of a controllable or uncontrollable emergency or the existence of an extreme inflow condition, post-earthquake condition. One Team member contact is all that is required to notify. Additional Team and/or District personnel will be alerted by the contacted Team member.

District Emergency Response Team

1. Chief, Emergency Management Branch (Team Chief)

2. Chief, Geotechnical Branch

   Alternate:
   Chief, Soils Design Section

3. Chief, Design Branch

   Alternate:
   Chief, Design Section A

4. Chief, Hydrology and Hydraulics Branch

   Alternate:
   Chief, Reservoir Regulation Section

5. Chief, Operations Branch

   Alternate:
   Chief, Engineering Evaluation Section

6. Dam Safety Coordinator

   Alternate
   Assistant Dam Safety Coordinator

The following list is to be used after the District Emergency Response Team is notified of a controllable or uncontrollable emergency, post earthquake condition, extreme inflow condition, or security alert.

The contacted member of the Emergency Response Team will call:

1. Chief, Engineering Division

   Alternate:
   Asst. Chief, Engineering Division

2. Chief, Construction-Operations Division

Alternate:
Asst. Chief, Construction-Operations Division

Chief, Engineering Division or Alternate will call:

Chief, SPD Emergency Management Branch

Alternate:
SPD Natural Disaster Manager or
SPD National Emergency Manager

Chief, Construction-Operations Division or Alternate will call:

District Engineer

Alternate:
Deputy District Engineer (Civ.)

District Engineer or Alternate will call:

Division Engineer

Alternate:
Deputy Division Engineer

APPENDIX C

APPENDIX C
SPECIAL DAM INSPECTION TEAM
NOTIFICATION LIST
SEVEN OAKS DAM

The following list is to be used to alert the Special Dam Inspection Team upon declaration of an extreme inflow, post-earthquake condition, or other emergency condition. One Team member is all that is required to contact to activate the Team. The contacted member will notify the other members.

Special Dam Inspection Team.

Project Engineer (Con-Ops), Team Chief

Asst. Project Engineer (Con-Ops), Member

Embankment Engineer (Engr), Member

Hydraulic Engineer (Engr), Member

Project Geologist (Engr), Member

Project Geologist, (Engr), Member

Embankment Inspector (Con-Ops), Member

Southern California Edison Company Hydropower Division

Alternate:

Soils Engineer

Engineering Geologist

APPENDIX C
EXTREME INFLOW INSPECTION CHECKLIST

Inspector________________ Date________________ Time____________

Pool Elevation____________________

Item	Yes	No	Description
1. Dam			
a. Crest			
(1) Misalignment			
(2) Settlement			
(3) Heaving			
(4) Cracks			
(5) Camber			
b. Upstream Face			
(1) Misalignment			
(2) Cracks			
(3) Slide(s)			
(4) Slope Protection Defect			
(5) Sinkholes			
(6) Settlement			
(7) Displacement			
(8) Reservoir Surface Disturbance (eddy, vortex, etc.)			
c. Downstream Face			
(1) Slide(s)			
(2) Signs of Movement			
(3) Cracks			

Item	Yes	No	Description
(4) Seepage			
(a) Location			
(b) Quantity			
(c) Clear or Turbid			
(5) Slope Protection Defect			
(6) Unusual Conditions			
2. Outlet Works Channel (Immediate vicinity of dam)			
a. Slope Protection			
b. Side Slope Stability			
c. Unusual Conditions			

APPENDIX D

APPENDIX D
EARTHQUAKE ALERT DATA SHEET

Upon receiving a telephone alert from the U.S.G.S. Earthquake Information Center, located in Denver, Colorado, (303) 236-1500, that an earthquake has occurred in the jurisdiction of the Los Angeles District, record the pertinent information by completing the blanks provided below.

Date:______________________________

Time Alert Call was Received:____________________

Time Earthquake Occurred:____________________

Earthquake Magnitude (Richter Scale):________________

Earthquake (Epicenter) Location:________________

Latitude:__________ Longitude:__________ State:__________

Nearest Community:____________________

Damage Assessment:____________________

____

Then evaluate the seismic event using: the pertinent Earthquake Evaluation Map located in this Appendix and the following criteria to determine if the earthquake was potentially damaging. An earthquake is to be considered potentially damaging if it occurs with a Richter magnitude of:

4.5 or greater within a 3-mile radius of the dam.

5.0 or greater within 30 miles.

6.0 or greater within 50 miles.

7.0 or greater within 70 miles.

8.0 or greater within 100-mile radius.

If the earthquake meets the criteria, immediately declare a Post-Earthquake Condition, notify: the Special Dam Inspection Team to activate using Appendix C, and the District Emergency Response Team using Appendix B.

APPENDIX D
POST-EARTHQUAKE INSPECTION CHECKLIST

Inspector________________ Date________________ Time________________

Pool Elevation__________ Earthquake Magnitude (Richter Scale)__________

Time of Occurrence__________ Distance from Epicenter (mi.)__________

Item	Yes	No	Description
1. Dam			
a. Crest			
(1) Misalignment			
(2) Settlement			
(3) Heaving			
(4) Cracks			
(5) Camber			
b. Abutment Contacts			
(1) Slides			
(2) Signs of Instability			
c. Upstream Face			
(1) Misalignment			
(2) Cracks			
(3) Slide(s)			
(4) Slope Protection Defect			
(5) Sinkholes			
(6) Displacement			
(7) Reservoir Surface Disturbance (eddy vortex, etc.)			
d. Downstream Face			
(1) Slide(s)			
(2) Signs of Movement			

Item	Yes	No	Description
(3) Cracks			
(4) Seepage*			
(a) Location			
(b) Quantity			
(c) Clear or Turbid			
(5) Slope Protection Defect			
(6) Unusual Conditions			
2. Spillway Approach, Floor, Crest, Training Walls			
a. Side Slope Stability			
b. Misalignment			
c. Cracks			
d. Exposed Reinforcement			
e. Settlement			
3. Spillway			
a. Misalignment			
b. Cracks			
c. Exposed Reinforcement			
d. Settlement			
4. Spillway - Outlet Channel (Immediate vicinity of project)			
a. Slope Protection			
b. Side Slope Stability			
c. Unusual Conditions			

* See page D-5.

Item	Yes	No	Description
5. Outlet Works - Trash Racks Recorder House and Shaft, Gate Structures, Access Galleries, Gate Chamber, Outlet Conduits, Stilling Basin			
a. Misalignment			
b. Cracks			
c. Exposed Reinforcement			
6. Outlet Works - Gates			
a. Visually check gates and hoisting mechanisms for signs of distress. If no impoundment exists, operate each gate through full cycle. If impoundment exists, do not exercise gates until receiving approval from the Reservoir Regulation Unit.			
7. Outlet Channel (immediate vicinity of dam)			
a. Slope Protection Defect			
b. Side Slope Stability			
c. Unusual Conditions			
8. Service Bridge			
a. Misalignment			
b. Cracks			
c. Exposed Reinforcement			
9. Instrumentation Monitoring			
a. Accelerographs			
o Activated (Lights on)			

Item	Yes	No	Description
b. Monitor and Record			
(1) Embankment Settlement Pipes			
(2) Conduit Settlement Points			

*If post-earthquake inspections are performed when a pool is being impounded, the following applies:

Since it may take time for a seepage condition(s) to occur, visually inspect for seepage immediately after the earthquake. If no new or increased seepage is detected, reinspect for seepage within 2-4 hours, 6-8 hours, 18-24 hours, and 48 hours after the earthquake unless otherwise instructed by District. If a new or increased seepage condition(s) is detected during any inspection after the earthquake, monitor and record seepage hourly until instructed otherwise by District.

APPENDIX E

APPENDIX E
INVENTORY OF DISTRICT BASE YARD RESOURCES

The Base yard
645 North Durfee Road
El Monte, California (213) 283-2757

Trucks:

1/2 ton, 4x2	29 ea
Pickup, 1/2 ton 4x4	1 ea
1/2 ton, 4x4, Utility "Bronco"	3 ea
Pickup, 3/4 ton, 4x2	5 ea
Pickup, 3/4 ton, 4x4	9 ea
Pickup, 1 ton, 4x4	2 ea
3/4 ton, 4x2, Comp Body	2 ea
3/4 ton, 4x4, Comp Body	1 ea
1 ton 4x2, Comp Body	2 ea
1-1/2 ton, 4x2	4 ea
2-1/2 ton, 4x2,	3 ea
2-1/2 ton, 4x2, Dump	1 ea
3 ton	1 ea
5 ton, Dump	2 ea

Vehicles:

Jeep, 1/2 ton, 4x4, CJ-5	2 ea
Van, 3/4 ton, 4x2, Utility	1 ea

Tractors:

Crawler, with winch, John Deere	1 ea
Crawler, with loader and 4 in 1 bucket Diesel-Driven	1 ea
Crawler, with loader and 4 in 1 bucket Diesel-Driven, "Case" 850	1 ea

Crawler, with loader and 4 in 1 bucket Diesel-Driven "IHC"	1 ea
Crawler, with disc and loader, Diesel-Driven, "Allis Chalmers"	1 ea
Farm-Type, Industrial Loader, wheeled Diesel-Driven, with rear blade, "IHC"	1 ea
Farm-Type, with loader, wheeled, Diesel-Driven, "Massey Ferguson"	1 ea
Wheeled, with loader and backhoe Diesel-Driven, "IHC"	1 ea
Wheeled, with mower, Diesel-Driven "IHC"	1 ea

<u>Special Equipment</u>

Trailers for hauling tractors, Tilt type	7 ea
Road Grader, Motorized, with Articulated frame, "John Deere"	1 ea
Emergency Trailer - Equipped with the following:	1 ea

Two-way radio, single side band, VHF FM radio, air-to-ground, scanner, UHF FM, microwave oven, range, refrigerator, storage space, sink, full bathroom, air conditioning, sofa can be used as bed.

<u>Electric Generators</u>:

Caterpillar generator, trailer mounted	1 ea

3 phase, 75 KVA, 60 KW, Power Factor 8
Low connection, 240 volts, 181 amps
High connection, 480 volts, 90 amps

Desco electric light unit with generator, trailer mounted	1 ea

120/240 volts, KW 7.5, amps 34

Iman electric light unit with generator, trailer mounted	1 ea

ac volts 120, 1 phase, 3.0 KVA, 3.0 TW, Power Factor 1, amps 25

Kohler portable generator	2 ea
120/240 volts, 3.5 KW, 1 phase	
Fairbanks-Morse electric light and power plant	1 ea
115 volts, ac watts 600, 1 phase, full load, amps 5.2	
Katolight	1 ea
KVA 125, KW 100, 3 phase, power factor 8, 277/480 volts, amps per terminal 150	
Emergency Trailer Generator	1 ea
120 volts, 1 phase, 120/240 volts, 1 phase 120 volts, 3 phase, 120/240 volts, 3 phase 240/480 volts, 3 phase	

APPENDIX E
SOURCES OF ASSISTANCE

This appendix lists vendors, contractors, and major equipment items possessed by each.

Procurement of emergency assistance from contractors, including rental of equipment is subject to policies and requirements prescribed in the Armed Services Procurement Regulation (ASPR), Army Procurement Procedures (APP), Engineer Contract Instructions (ECI) identified as ER 1180-1-1, and the provisions of ER 500-1-1, Emergency Employment of Army and Other Resources: Natural Disaster Procedures and the NPS supplement to ER 500-1-1. In the event emergency assistance must be obtained during duty hours, immediately contact the District Chief of the Procurement Branch (213) 894-5660. During off-duty hours, the following personnel should be contacted at home for assistance:

Chief, Contracting Division

Chief, Procurement Branch

Chief, Contract Compliance Branch

APPENDIX E
LIST OF SUPPLIES AND EQUIPMENT

Firm Name and Address	Telephone	Items
1. Vendors of Supplies		
An-Wil Inc. 1700 S. Santa Fe. Ave. Los Angeles, CA 90021 Wm. Steinman	624-0104 Emergency: 838-1104	Burlap bags--300,000 Polypropylene--150,000 to 200,000
Atlas Wire and Cable Corp. 133 S. Van Norman Montebello, CA 90640 Sales Dept.	723-2401	Communications Cable, Tubing, Wire
Baron Bag Co. 156 S. Anderson St. Los Angeles, CA 90033 Barry Tzinberg Mrs. Gould Larry Gould	268-8458 Emergency: 886-9466 652-1647 826-7682	Burlap Bags--40,000 to 300,000
Central Bag and Supply Co. 2222 E. Olympic Los Angeles, CA 90021 Tom Sepik	624-4242 Res. 371-7257	Can furnish 100,000 polyethylene bags on 48-hr notice (48-hr partial delivery-20,000 daily subject to size of bag)
Chase Bag Co. 4707 E. 49th St. Vernon, CA 90058 Jim Watts	582-8211	Can manufacture polyethylene, burlap, and polypropylene 25,000 to 30,000 per day up to 60,000. Has information on emergency stocks.
Federal Stock USMC Logistics Support Base Pacific Barstow Barstow, CA 92311 John Ford Cmdr R. Santi FTS 8-894 Battalian Duty Officer - After duty	 (619) 577-6547 (619) 235-3541 (619 577-6611	Burlap bags
L.S. Hawley Corp. 5277 N. Vincent Ave. Irwindale, CA 91706 Dave Hartley	334-1208 Emergency: (714) 599-3353	Rock Rip Rap from Quarries in Lompoc, Santa Barbara, Camarillo, Corona, San Jacinto and San Diego.

Firm Name and Address	Telephone	Items
Di Cecco's Bldg. Ctr. 21355 Sherman Way Canoga park, CA 91303 Vince Di Cecco	340-2400 Res. 884-6192	Lumber and timbers; finished lumber, plywood, hardware, building materials (Plumbing and Elec.) power tools
Friedman Bag Co. 801 E. Commercial St. Los Angeles, CA 90012 Mr. Wesler Mr. Lanfeld Mr. Moss	628-2341 Res. 877-2317 Res. 553-2677 Res. 782-4995	Burlap bags (100,000)
McLaughlin Industrial Distribution Inc. 7141 Paramount Blvd. Pico Rivera, CA 90660 Dominic Rounds E. H. McLaughlin, III	723-2411 Res. 721-0650 Res. 441-4903	Small tools, Rope, shovels, Hardware supplies.
Motorola Communications and Electronics 2333 Utah El Segundo, CA 90245	536-0700 Res. 345-7635	Portable Radio facilities, FM 2-way radio systems.
Paco-Pacific Pumping Co. 6838 Acco St. Commerce, CA 90040	685-3250	Pumps
Sierra Bag Co. P.O. Box 1340 Bakersfield, CA 93302 Ben Sacco - Morris Rosenberg	(805) 327-9386	Burlap Bags (100,000-500,000)
Southwestern Bag Co. 1380 E. 6th St. Los Angeles, CA 90221 Milton Baran	622-6122	Burlap Bags (100,000), Plastic sheeting, Pressure sensitive tape

2. Equipment

a. Construction Equipment

Associated General Contractors of America, Los Angeles Dist. 2551 Beverly Blvd. Los Angeles, CA 90057 District Mgr.-Gary Butler	385-6031 Res. 889-5192	Emergency general construction equipment for disaster relief and rehabilitation

Firm Name and Address	Telephone	Items
M.P. McCaffrey, Inc. 2121 E. 25th St. Los Angeles, CA 90058 Tom McCaffrey Mike McCaffrey T.J. Koetters	588-7181 Res. 283-1985 Res. 373-7842 Res. 351-0941	Truck cranes, crawler, cranes and draglines, hydraulic cranes, road graders, crawler and rubber-tired front-end loaders, hydraulic backhoes, clamshell and dragline buckets, rock grapples, off-highway end dump trucks.
Maritime Power Corp. 39 Broadway, Rm. 3300 New York, NY. 10006 Mike Nahon	(212) 422-3967	Stocks all types of centrifugal and reciprocating pumps; motors and turbines to drive this equipment from 10HP to 1000Hp. Diesel Engines an generators. Quick delivery.
Owl Crane & Rigging Co. 500 S. Alameda St. Compton, CA 90224 or 10641 Mulberry Ave. Fontanna, CA 92335	636-9921 638-8761 (714) 823-0695	Truck cranes 30-165 ton, Hydraulic cranes 25-75 ton, Crawler cranes to 225 ton

b. Helicopters-Civil

Firm Name and Address	Telephone	Items
Briles Helicopter Service 16450 Arminta St. Van Nuys Airport Van Nuys, CA 91406 Joe parr	994-1445 Res. 374-4519	Helicopter: 3 206's (4 pass); 1 Silkorsky S58T (16 pass); 2 206L-1 (6 pass); 1 206L-3 (6 pass)
National Helicopters Service and Engineering Service 16800 Roscoe Blvd. Van Nuys, CA 91406	345-5222 24 Hr service	Helicopters: (1) 47G2, (1) 47J2; and (25) 4G3B1; (3) Bell Jet ranger 206B
Western Helicopter, Inc. 1670 Miro Way Rialto, CA 92376 Dorcey Wingo	(714) 874-1222	Helicopters: 1 Bell 205A Model 14-pass; Jet ranger (4 pass); U500 (4 pass); LAMA (4 pass); 319B (6 pass); Hughes 300C (2 pass; 1 Beach BE-76 Duchess (3 pass); 1 Beech BE-77 Skipper (1 pass); 1 Cessna 182A Skkyhawk (3 pass)

Firm Name and Address	Telephone	Items
Riverside Air Service 6741 Flight Rd. Riverside, CA 92504	(714) 689-1160	BE-90 Twin (7 pass); PA 32R (5 pass); BE 76 Twin (3 pass); B55 Twin (3 pass); 1 LR24 Learjet (6 pass)
Helitac Aviation Inc. 1910 W. Sunset Blvd. Suite 900 Los Angeles, CA 90026 Mike Dreesman	483-6898 24 Hr service	Helicopters: 1 Hughes 500 C (HU 369 HS); seats pilot plus 3 pass. 1 Bell Jet-ranger 206B; seats pilot plus 3 to 4 pass. 1 Bell 206L Longranger; seats pilot plus 5 pass.

c. Fixed-Wing Aircraft

G and H Aircraft P.O. Box 4819 El Monte, CA 91731 Carol Nahom	442-2930	Cessna 182, 230 HP, 4 place; Beechcraft Debonair, 285 HP, 4 place; Piper Lance, 300 HP 6 place; Cessna 310, 2x260 HP, 6 place; Beechcraft Baron, 2x260 HP, 5 place; Piper Cheyenne, 2x500 HP, 7 place
General Aviation Co. 3915 W. Commonwealth Fullerton, CA 92633 Bill Gant	(714) 526-6611	Piper PA 28, 150 HP, (4 pass); Piper PA 28, 180 HP, (4 pass); Piper PA 32, 300 HP, (6 pass); Piper PA 34, 400 HP, (6 pass)
Pacific Executive 2733 E. Spring Long Beach, CA 90806	595-4353 (714) 821-9171	

3. Contractor yards

L.S. Hawley Corp. 5277 N. Vincent Ave. Irwindale, CA 91706 Dave Hawley Jim Hawley	334-1208 Res. 334-5383 Res. 334-2683	2 dozers, 1 swamp cat, 2-15 yd. semi-dumps, 3 graders, 14 loaders, 3 cranes, 10 pick-ups w/two-way radio.

Firm Name and Address	Telephone	Items
Guy F. Atkinson Berth 115, LA Harbor San Pedro, CA 90731 Joe Beck, Equip. Svcs. Res.	775-1281 834-2607 (714) 595-5581	Trucks, cranes, shovels, tractors, scrapers, pumps, compressors, draglines with operators and misc. construction materials. Barges & marine equipment.
A.J. Diani Construction Co. 295 North Blosser Road P.O. Box 636 Santa Maria, CA 93456	(805) 925-9533 After hours: (805) 922-2079	Dozers, tractors, loaders, trucks, lowbeds.
Jess Hubbs and Sons P.O. Box 490 Rialto, CA 92376 Don Hubbs	(714) 875-1161 (714) 862-1798	7 dozers from D-5 to 834, 4-633 scrapers, 2 Blades, S-7 & TS-14 water pulls, 980 loader, 4 TS-24 Scrapers
Paul Hubbs Construcion Co. 140 W. Valley Blvd. Rialto, CA 92376 Paul Hubbs Res. Jay Hubbs Res.	(714) 877-2726 (714) 825-2236 (714) 884-1762	Various equip. incl: Dozers, Dumps, Trucks Loaders, Scrapers, Cranes, Compressors, Light plants, Scales, Pumps, Backhoes, Generators, etc.
Kruse Construction Co. 11300 Pendleton St. Sun Valley, CA 91352 Robert Kruse Res.	875-3030 244-2528	Trucks, cranes, shovels, tractors, scrapers, pumps, compressors, graders, loaders, backhoes, misc. const. material
Matich Corp (Yard) 1371 S. LaCadena Colton, CA 92324 Martin Matich Res. R.A. Hays Res.	(714) 877-2100 (714) 886-5285 (714) 888-4243	Trucks, cranes, tractor, draglines, graders, loaders w/ operators, asphalt equip., Riprap Quarry Stone.
Oberg Construction 645 Cockron Res. P.O. Box 747 Simi Valley, CA 93062 Mr. Oberg	883-9390 (805) 985-8642	Crawler, dozers, motor patrols, tractors, compressors with operators, and misc. steel and timber beams.
E.L. Yeager Construction Co. P.O. Box 87 Riverside, CA 92502	(619) 684-5360	Dozers, tractors, trailers, lowbeds, welders, dumps.

Firm Name and Address	Telephone	Items
The URSA Corp. 3045 Rosecrans St., Suite 310 San Diego, CA 92110 Abel Parra	(619) 222-2492 Res. (619) 474-3922	Various equip. incl: dozers, scrapers, swamp dozers, blades, pumps, track backhoes and light towers; varied tech. personnel; licensed explosives contr.; 24 hr. availability.
J Harris Co. P.O. Box 941 Chino, CA 91710 John Harris	(714) 947-3908	Kamatsu Sump Cats and other various equip.

4. Military Helicopters. During flood emergencies or other natural disasters, military helicopters may be required. Service must be coordinated with the Emergency Management Branch, Ext. 3440 and the Management Analysis Branch, Ext. 5350.

Los Angeles Area:

Armed Force Reserve Ctr. Army Aviation Support Facility California National Guard Bldg. 43 AFRC Los Alamitos, CA 90720	596-5526 Ext. 524 LTC Chormlev Maj. Fletcher	UH1H (10 pass); Bell OH58 (pilot + 3 pass)
63rd ARCOM ASF Bldg. 42 Los Alamitos AFRC Los Alamitos, CA 90702	430-7418 Autovon 972-8502 A. Karas 956-4917 Maj. John Nass 222-5147	UH1H Huey (10 pass); U21A Fixed Wing (6 pass)
Commanding Officer US Coast Guard Air Sta. 5885 W. Imperial Hwy. Los Angeles, CA 90045	FTS 8-966-5012 642-5012	(3) H52A Sikorsky (4 pass)

San Diego Area:

Commanding Officer US Coast Guard Air Sta. 2710 Harbor Drive San Diego, CA 92101 CDR Post	FTS 8-895-5870* (619) 293-5870* Emerg. (619) 295-3121* *24 Hr. Number	(3) HH-3F Sikorsky (10 pass)

Firm Name and Address	Telephone	Items
5. Bailey - type Bridges (lease or purchase)		
Bailey Bridges, Inc. 201 Bridge St. P.O. Box 1186 San Luis Obispo, CA 93406 Rick Hamlin, Pres. Res. Fred Hamlin Res.	(805) 543-8083 (805) 541-1203 (805) 543-1731	Bailey bridges - pre-fabricated
6. Borrow Materials		
Aman Brothers Materials 6701 South Central Los Angeles, CA	582-6445	Rock
Conrock Co. 3250 San Fernado Road Los Angeles, CA	(213) 258-2777	Sand & Rock
Paul J. Hubbs Const. 140 W. Rialto, CA	(714) 877-2726	Sand, Gravel, Rock, Riprap
Livingston-Graham 16080 E. Arrow Way Irwindale, Ca	(213) 681-7101	Sand & Gravel
Pacific Rock & Gravel Co. 1465 E. 16th St. Upland, CA	(714) 985-7283	Rock, Gravel

APPENDIX F

APPENDIX F
CORRECTIVE ACTION INFORMATION
SEVEN OAKS DAM

Wave Damage and/or Erosion of Dam Upstream and Downstream Faces of Embankment

a. Potential Problems

Wave damage may occur during periods of high northerly winds. Damage may include erosion of the underlying materials causing collapse of the stone protection. Wave damage is particularly serious during abnormally high reservoir pool levels when serious erosion can cause a sudden collapse of the crest with subsequent overtopping of the embankment. The downstream face of the embankment is also subject to erosion due to runoff from heavy rains and waves breaking over the top of the embankment.

b. Corrective Action

The type of corrective action that is appropriate depends on the severity of damage, rate of progression of damage, and urgency of action. Temporary protection above and within 10-12 feet below the waterline can be provided quickly by use of plywood sheets, prefabricated panels or canvas as shown in figure 2-2 or by filling eroded areas with sandbags. Tables 2-1 through 2-4 provide information useful in estimating the amount of personnel and materials required. Protection further below the water level can be provided by dumping stone protection in the affected area. In cases of severe erosion, lowering of the reservoir pool level can shift wave forces to a lower elevation. Repairs normally require reconstruction of the eroded slope and replacement of both bedding materials and stone protection. Lowering of the pool level is usually required prior to making permanent repairs on the upstream face of the dam.

Table F-1. Approximate Requirements for Erosion Protection with Plywood.

Length To Be Protected	No Plywood Sheets Req'd	No. Stakes Req'd	No. Sandbags Req'd	Personnel Req'd	Hours To Complete
10	3	8	15	6	1.5
20	5	13	25	6	2.5
30	8	20	40	6	3.0
40	10	25	50	6	3.5
50	13	33	65	6	4.0
60	15	38	75	6	5.0
70	18	45	90	10	3.5
80	20	50	100	10	3.5
90	23	58	115	10	4.0
100	25	63	125	10	4.5
150	38	95	190	16	5.0
200	50	125	250	16	4.0
300	75	188	375	20	6.0
400	100	250	500	24	6.0

Table F-2. Approximate Requirements for Erosion Protection with Prefabricated Panels.

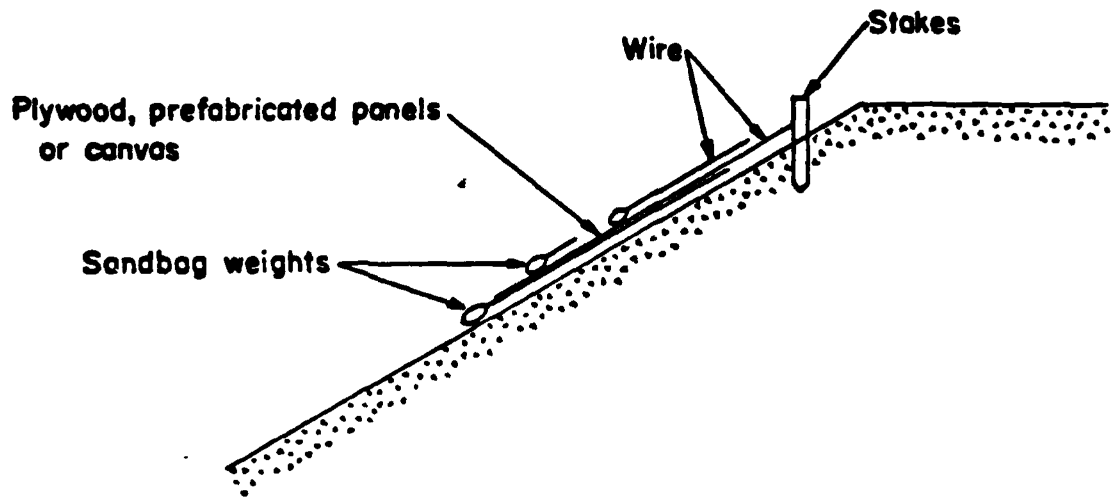
Length to Protect (ft)	100		200		300		400		500		1000	
No. Panels Req'd (@ 16' ft)	7		13		19		25		32		63	
Panel Width (ft)	3	5	3	5	3	5	3	5	3	5	3	5
Length 1" x 12" Req'd. (ft)	340	360	652	1170	900	1700	1200	2250	1500	2900	3000	5700
Length 1" x 6" Req'd. (ft)	80	140	160	260	230	380	300	500	400	640	750	1300
Stakes Req'd.	30	30	60	60	90	90	115	115	150	150	300	300
Sandbags Req'd.	35	60	70	100	100	150	125	200	160	250	320	500
Time to Complete	3	4.3	4.3	5.3	2.9	3.6	3.6	4.7	4.5	5.8	5.6	7.3
No. Personnel	8	8	8	8	16	16	16	16	16	16	24	24

Table F-3. Approximate Requirements for Erosion Protection with Canvas.

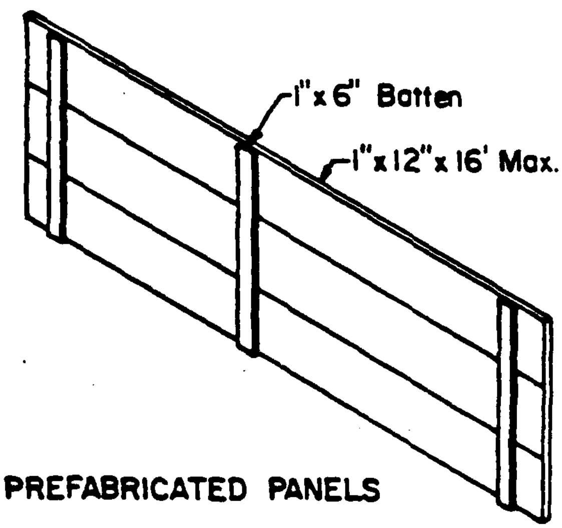
Length To Be Protected	Length Canvas Req'd.	No. Stakes Req'd.	No. Sandbags Req'd.	Personnel Req'd.	Hours to Complete
10	35	15	30	6	1
20	50	20	40	6	1.5
30	80	35	60	6	2.0
40	100	40	70	6	2.5
50	130	55	100	6	3.0
60	150	65	110	6	3.5
70	160	70	120	6	4.0
80	190	85	150	6	4.5
90	210	90	160	10	3.3
100	350	100	180	10	3.5
150	400	150	275	10	4.8
200	500	200	350	16	4.3
300	700	300	520	20	4.8
400	1000	400	700	24	5.0

Table F-4. Approximate Requirements for Filling Areas with Sandbags.

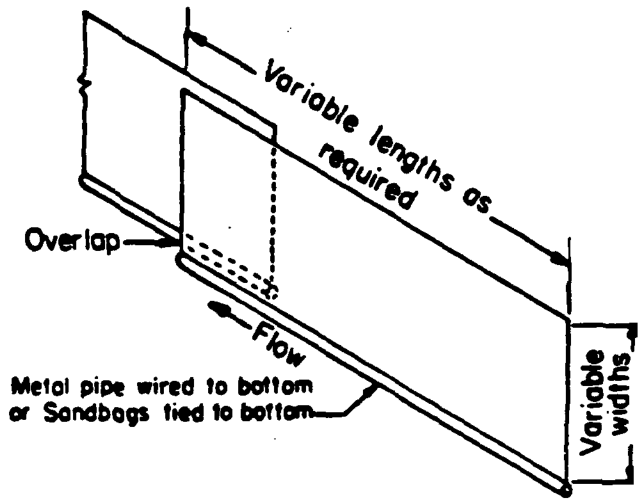
Volume (ft^3)	No. Bags Req'd.	Personnel Req'd.	Hours to Complete
100	250	6	1.5
200	500	6	3
300	700	6	4
500	1500	10	3.5
1000	2500	10	6.5
2000	4700	14	8.5
3000	7000	24	8
4000	9500	34	7.5
5000	11,700	38	8



GENERAL SCHEME FOR
TEMPORARY EROSION PROTECTION
FIGURE F-1



PREFABRICATED PANELS



COTTON BAGGING OR CANVAS

APPENDIX G

APPENDIX G
EXAMPLE STATEMENT FOR MEDIA NOTIFICATION
OF
IMMINENT OR ACTUAL DAM FAILURE

At _____(Time)_____ today, at Seven Oaks Dam near the city of Mentone California. ___(Describe Problem)___. The ___(Surrounding Areas)___ can expect ___(Problem)___.

Approximately ___(Amount)___ of water was contained behind the dam at the time of the dam failure. This water is entering the Santa Ana River flood plain at a rate of ___(Number of)___ cubic feet per second.

Federal, state, and local governmental agencies have been notified of the situation by Corps of Engineers, Los Angeles District officials, and SB, OC advised to begin evacuating affected areas immediately.

Radio and television stations ___(Name)___ in ___(Place)___ have been or are being notified of the situation and are airing bulletins concerning the flooding situation.

Corps of Engineers technical personnel are at Seven Oaks Dam and are initiating remedial measures.

Further information will be made available as it develops. The media contact at the Corps of Engineers during this emergency situation is (Person/Dept./Tele.No.) or inquiries may be made to the Public Affairs Office, (213) 894-5320.

APPENDIX G
EXAMPLE STATEMENT FOR MEDIA NOTIFICATION
OF
POTENTIAL DAM FAILURE

Engineers of the Corps of Engineers, Los Angeles District, SB, OC, have detected ___(Describe Problem)___ at Seven Oaks Dam near the city of Mentone California. As a result, the Corps of Engineers has initiated procedures to notify appropriate governmental officials at the Federal, state and local levels.

At this time, te situation is <u>not</u> critical. However, a team of engineers from the Los Angeles District headquarters.

At this time, the situation is <u>not</u> critical. However, a team of engineers form the Los Angeles District headquarters are on their way to the Seven Oaks Dam to investigate and evaluate the ___(Problem)___ and the potential effects it could have on ___(Place)___. The engineers will also evaluate corrective measures to be taken.

The Los Angeles District is monitoring the situation and will keep officials in ___(Place)___ appraised of developments. News media in ___(Place)___ will be kept up-to-date on the situation as well.

Because of the potential for serious downstream damage from a failure of the dam, local officials should have evacuation plans ready for immediate implementation should the ___(Problem)___ at Seven Oaks Dam become severe.

Based on availabe data concerning the ___(Problem)___, the stiuation is <u>not</u> critical. Further information will be made available through the District's Public Affairs Office as it develops.

Editor's Note: An example fact sheet on Seven Oaks Dam is atttached.

EXAMPLE FACT SHEET
FOR
SEVEN OAKS DAM

Pertinent Project Data

Drainage Area

Area of Reservoir (at spillway crest)

Capacity (at spillway crest)

Dam Type

Dam Height

Dam Length

Spillway Type

Uncontrolled Outlets - Number and Size

Controlled Outlets - Number and Size

Gates-Type

Regulated Discharge at Spillway Crest

Reservoir Design Inflow Peak

Spillway Design Inflow Peak

Description of Problem:

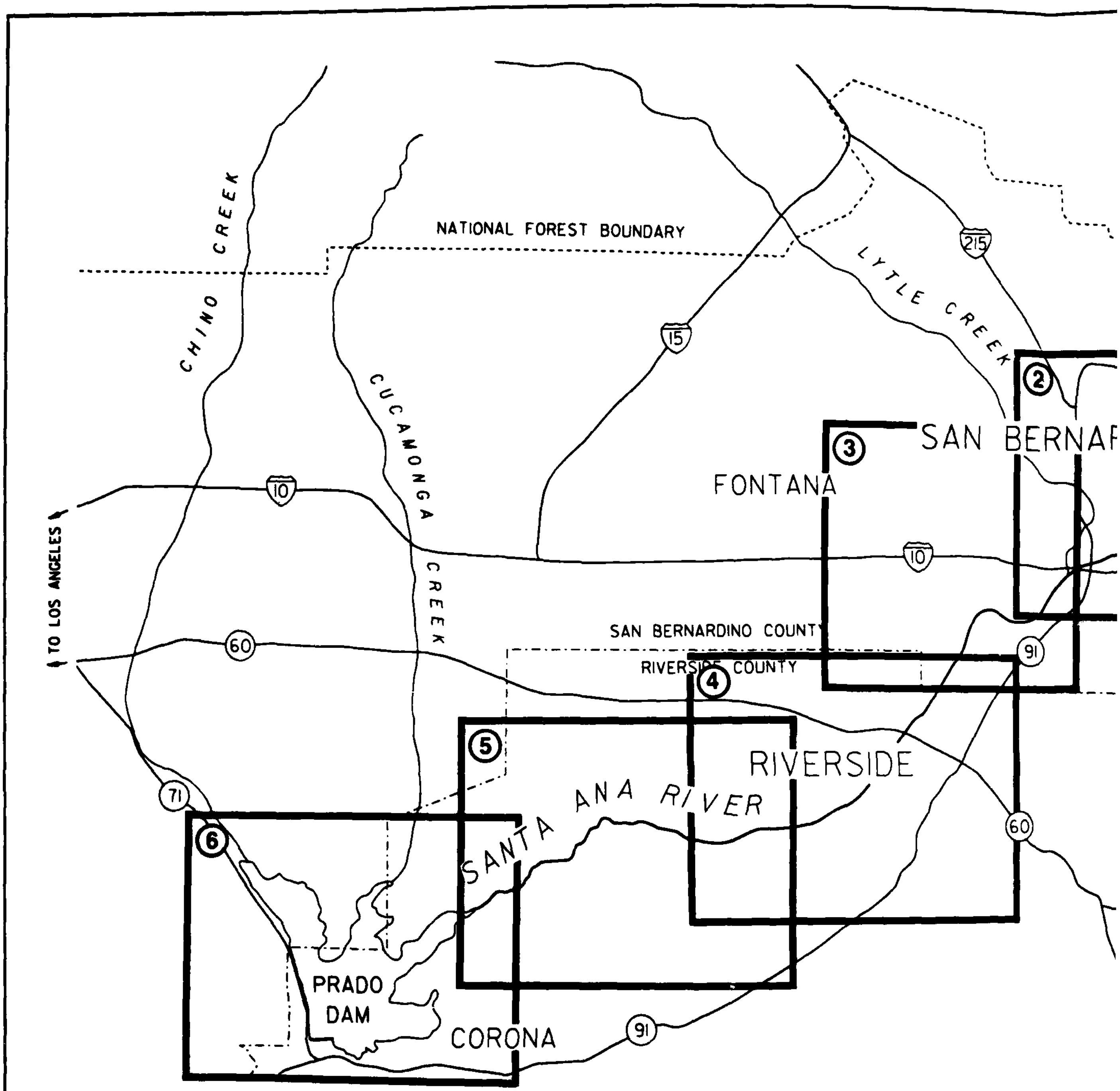
Present Situation Status:

Planned Actions:

Personnel on Site:

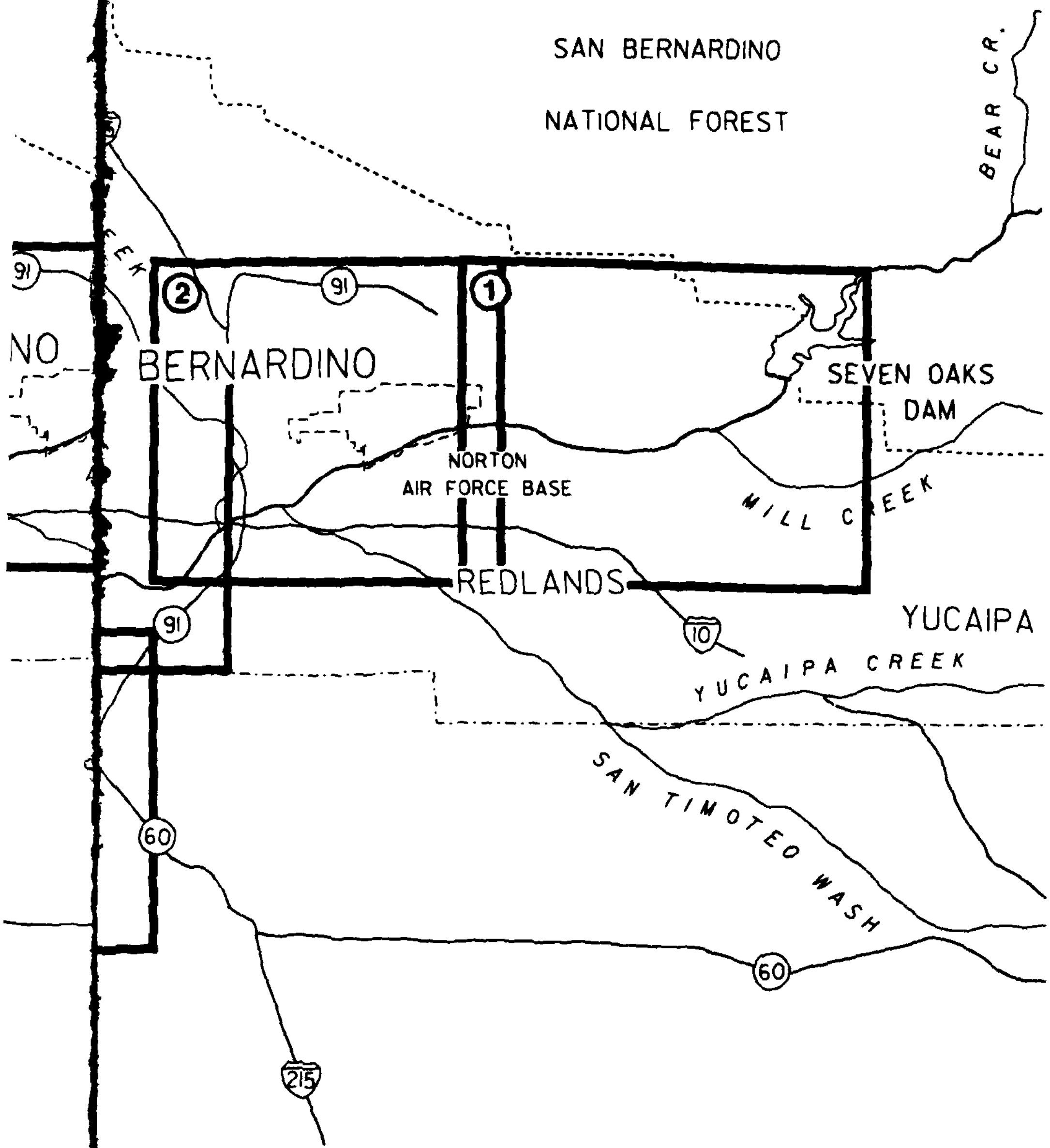
Person to Contact for Further Information:

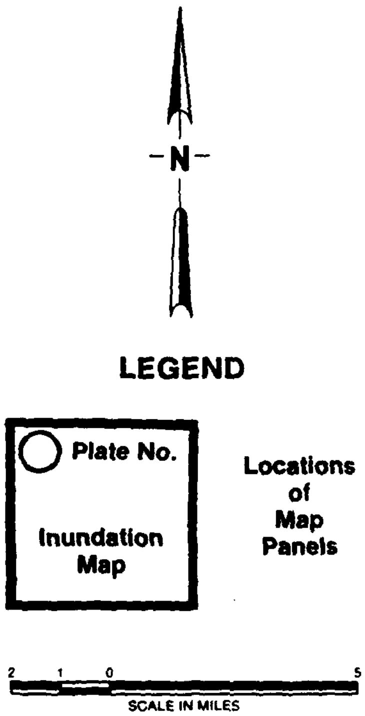
APPENDIX H



50 Foot Cofferdam, Pool Elevation 2150 Feet, NGVD

Section Number	Plate Number	Distance from Dam (Miles)	Invert Elevation (Feet, NGVD)	Arrival Time (Hours)	Peak Elevation (Feet, NGVD)	Time of Peak Elevation (Hours)	Avera
Cofferdam	1	N/A	2100	N/A	2150	0.06	
5	1	0.7	1940	0.1	1960	0.1	
10	1	3.8	1490	0.4	1500	0.5	
13	1	7.0	1218	2.0	1224	2.1	
15	2	8.8	1112	2.6	1118	2.7	
20	2	12.9	959	4.0	966	4.5	
23	3	16.7	861	5.0	868	5.6	
26	4	21.1	758	5.5	764	6.2	
28	4	23.7	698	5.9	705	6.8	
30	5	26.7	646	6.7	652	7.6	
33	5	30.0	559	7.5	565	8.6	
35	6	32.9	514	8.9	519	10.0	





ank

on GVD)	Time of Peak Elevation (Hours)	Average Overbank Depth (Feet)
	0.08	N/A
	0.1	0
	0.5	0
	2.1	0
	2.7	0
	4.5	0
	5.6	0
	6.2	0
	6.8	0
	7.6	0
	8.6	0
	10.0	0

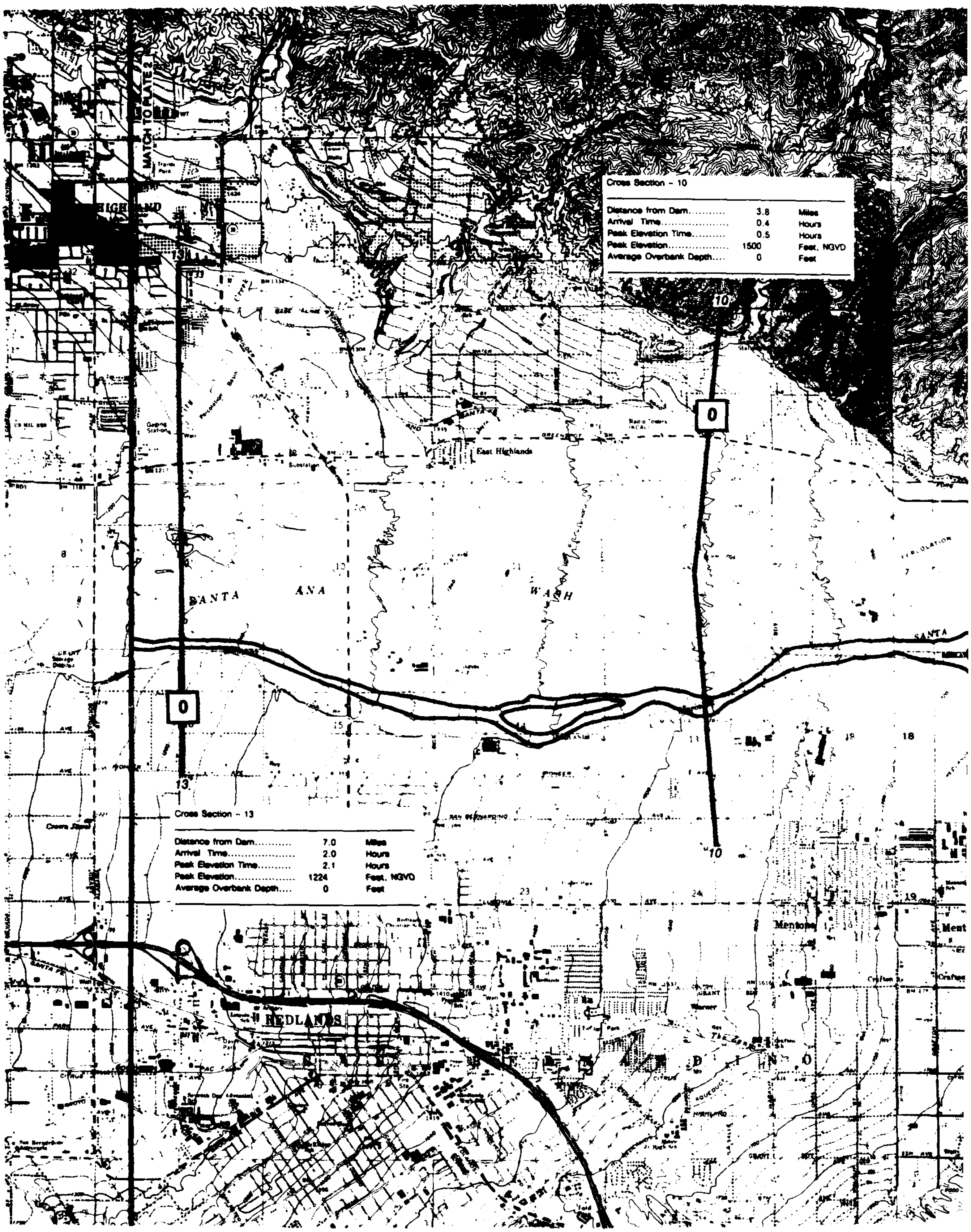
THIS MAPPING IS BASED ON THE HYPOTHETICAL FAILURE OF THE 50' CONSTRUCTION COFFERDAM.

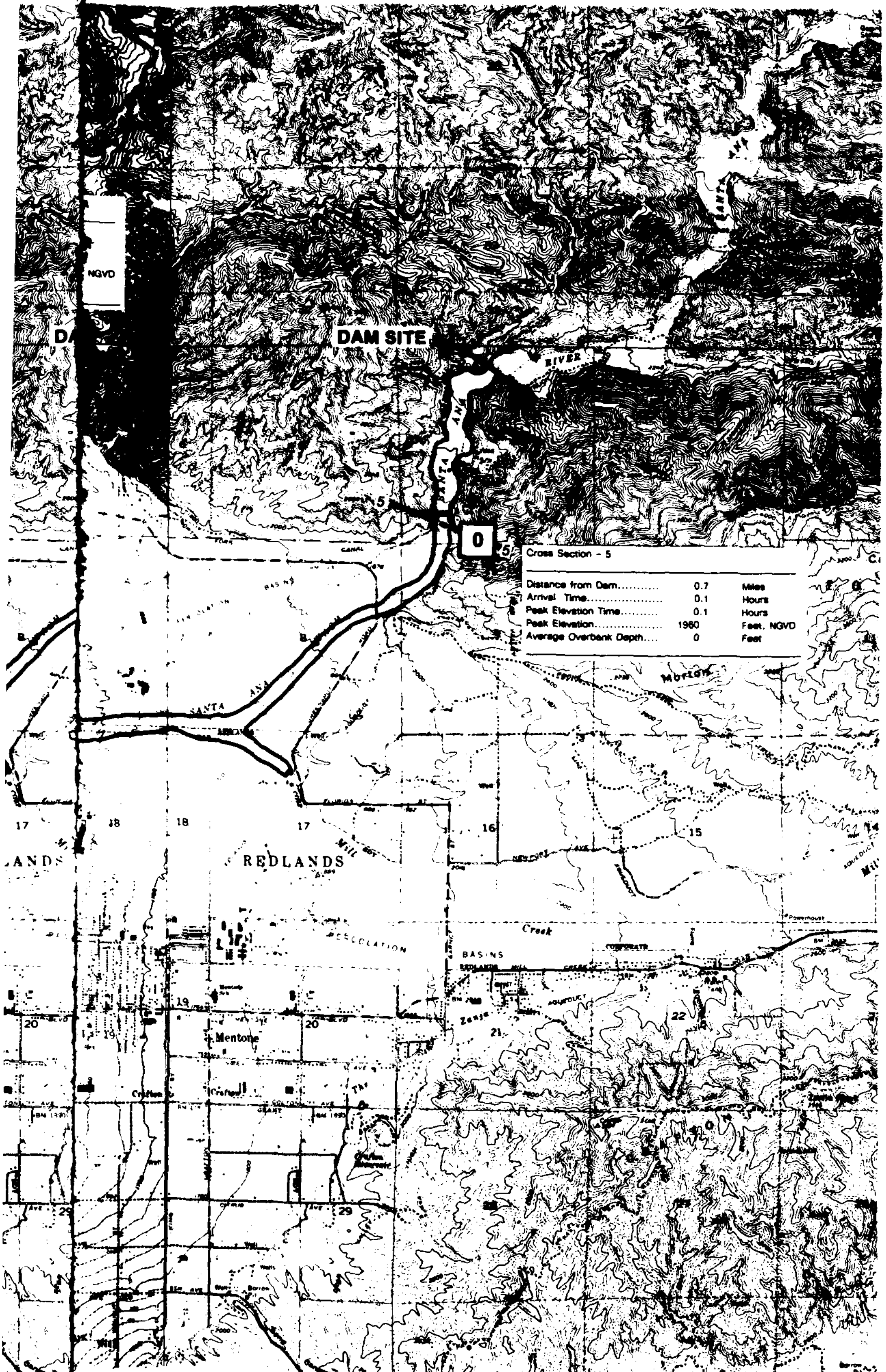
U.S. ARMY ENGINEER DISTRICT, LOS ANGELES
CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

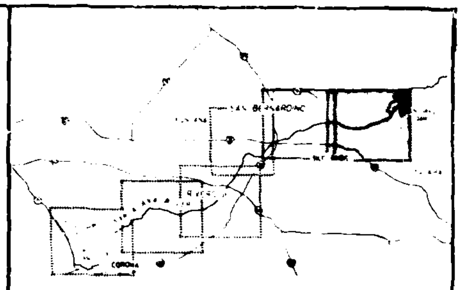
SEVEN OAKS COFFERDAM EMERGENCY PLAN

INDEX MAP

JUNE 1988


MATCH TO PLATE 2
HIGHLAND
Cross Section - 10
Distance from Dam.......... 3.8 Miles
Arrival Time.......... 0.4 Hours
Peak Elevation Time.......... 0.5 Hours
Peak Elevation.......... 1500 Feet, NGVD
Average Overbank Depth.... 0 Feet
10
0
East Highlands
SANTA ANA WASH
0
13
Cross Section - 13
Distance from Dam.......... 7.0 Miles
Arrival Time.......... 2.0 Hours
Peak Elevation Time.......... 2.1 Hours
Peak Elevation.......... 1224 Feet, NGVD
Average Overbank Depth.... 0 Feet
10
Mentone
REDLANDS





-N-

LEGEND

Limit of Flooded Area With Dam Failure

Cofferdam

0 Average overbank depth in feet

26 —— 26 Cross Section

2000 0 2000 4000 6000

SCALE IN FEET

National Geodetic vertical datum of 1929.
Source of base map: U.S. Geological Survey 7.5 minute series.
Contour interval 20 feet.
Redlands, 1967, photo revised 1980
Contour interval 40 feet.
Harrision Mtn., 1967, photo revised 1980
Keller Peak, 1967
Yucaipe, 1967, photo revised 1980

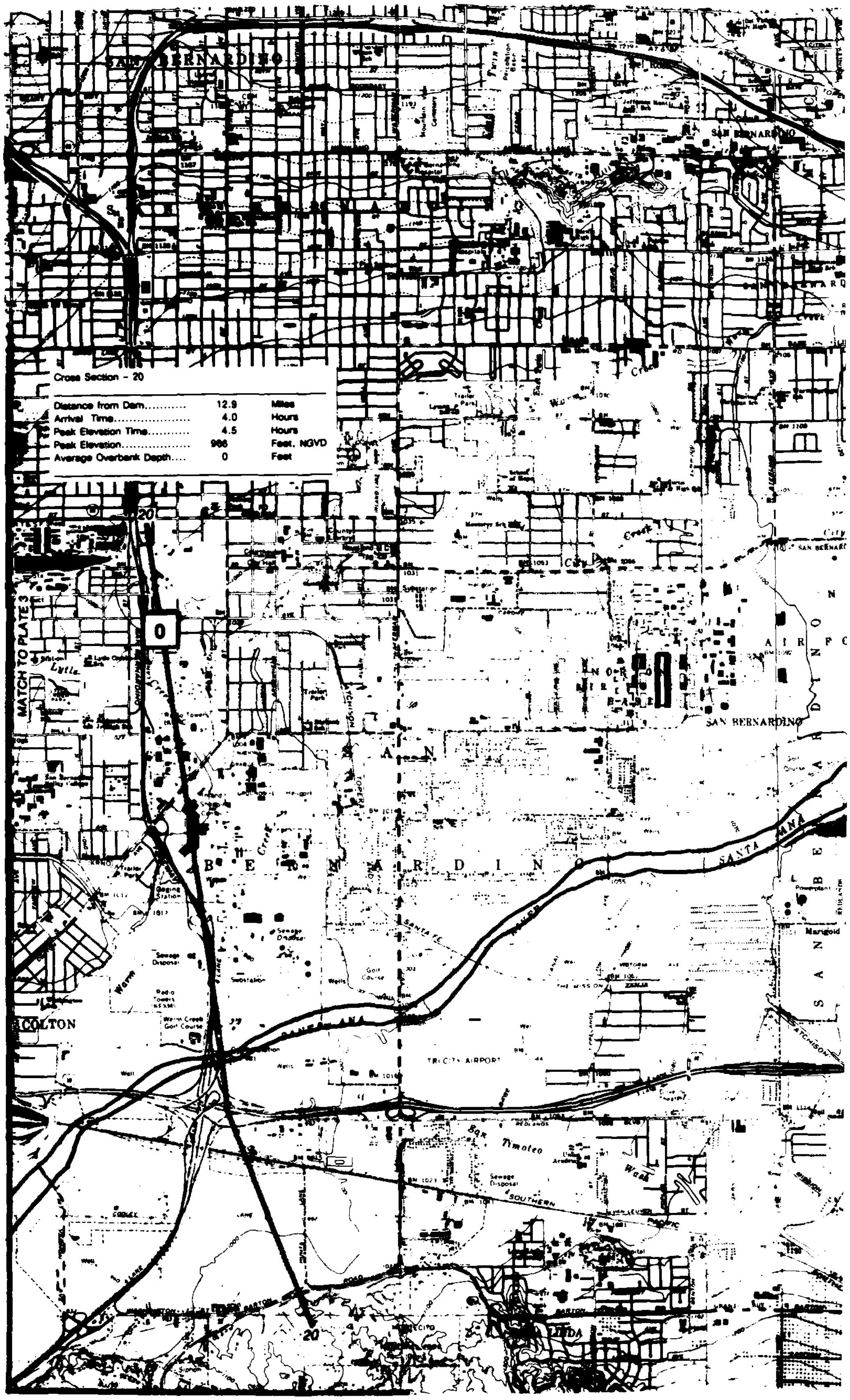
NOTE: The inundated areas shown on this map reflect potential flooding in the event of a cofferdam failure during construction of Seven Oaks Dam. The cofferdam was assumed to be 50 feet high.

U.S. ARMY ENGINEER DISTRICT, LOS ANGELES
CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

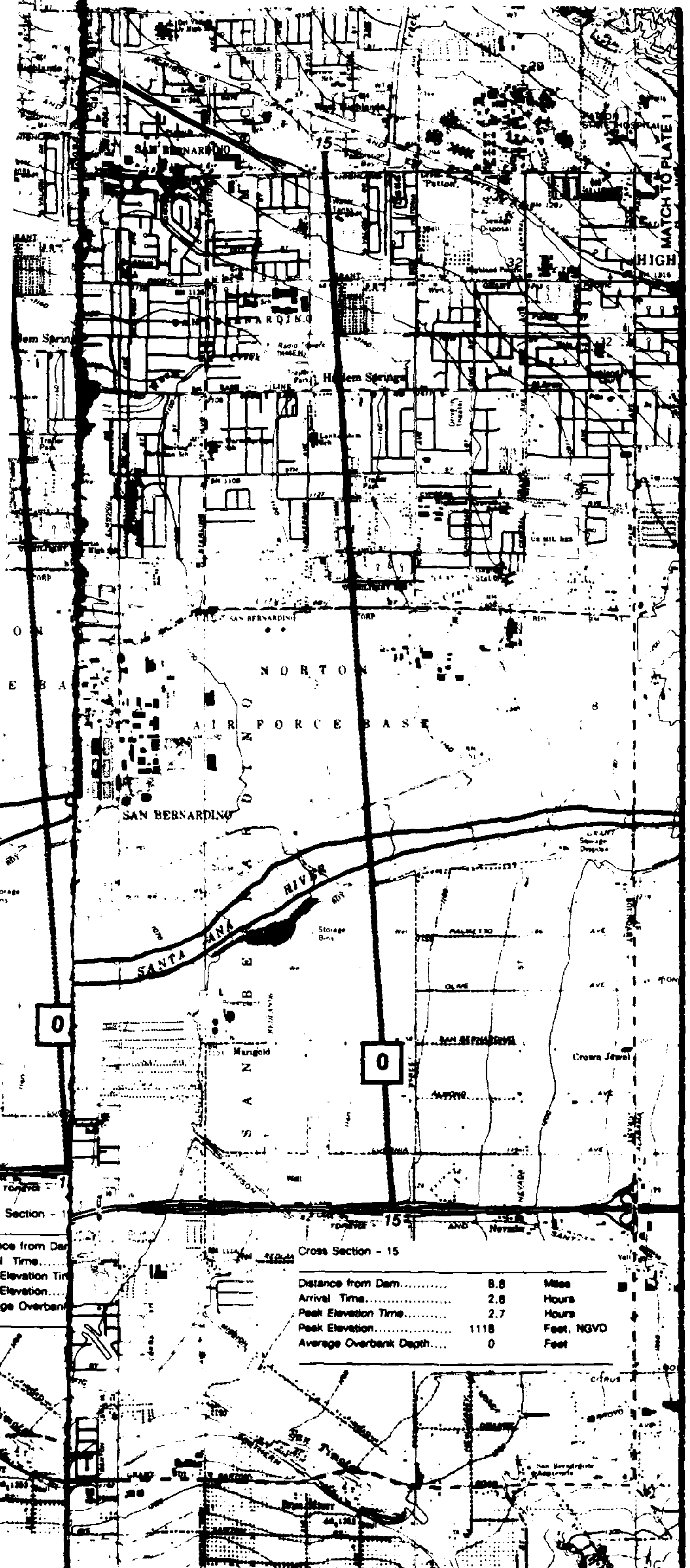
SEVEN OAKS COFFERDAM EMERGENCY PLAN

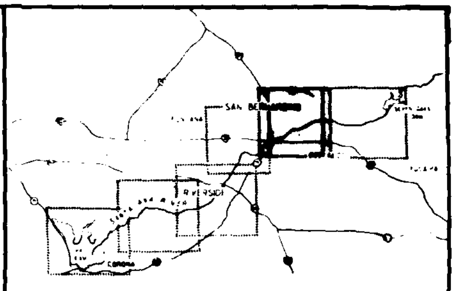
INUNDATION MAP

JUNE 1988 PLATE NO. 1



Cross Section - 20
Distance from Dam.......... 12.9 Miles
Arrival Time.......... 4.0 Hours
Peak Elevation Time.......... 4.5 Hours
Peak Elevation.......... 986 Feet, NGVD
Average Overbank Depth.... 0 Feet
MATCH TO PLATE 3
SAN BERNARDINO
COLTON
TRI CITY AIRPORT
SANTA ANA
20
0





-N-

LEGEND

Limit of Flooded Area With Dam Failure

Cofferdam

0 Average overbank depth in feet

26 ——— 26 Cross Section

2000 0 2000 4000 6000

SCALE IN FEET

National Geodetic vertical d[illegible]n of 1929
Source of base map: U.S. Geological Survey 7.5 minute series
Contour interval 20 feet.
Redlands, 1967, photo revised 1980
San Bernardino South, 1967, photo revised 1980
Contour interval 40 feet.
San Bernardino North, 1967, photo revised 1980
Harrison Mtn., 1967, photo revised 1980

NOTE: The inundated areas shown on this map reflect potential flooding in the event of a cofferdam failure during construction of Seven Oaks Dam. The cofferdam was assumed to be 50 feet high.

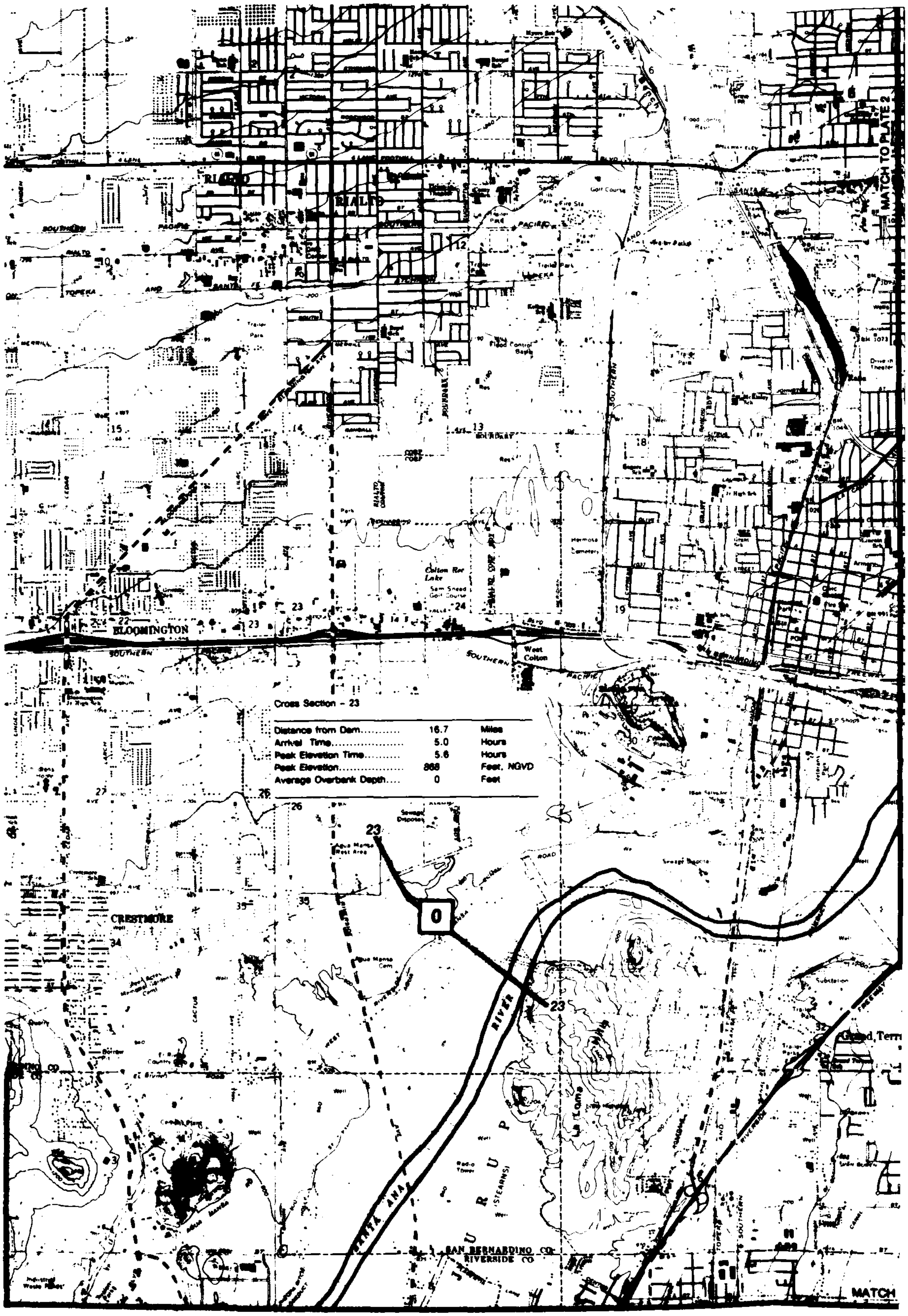
U.S. ARMY ENGINEER DISTRICT, LOS ANGELES
CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

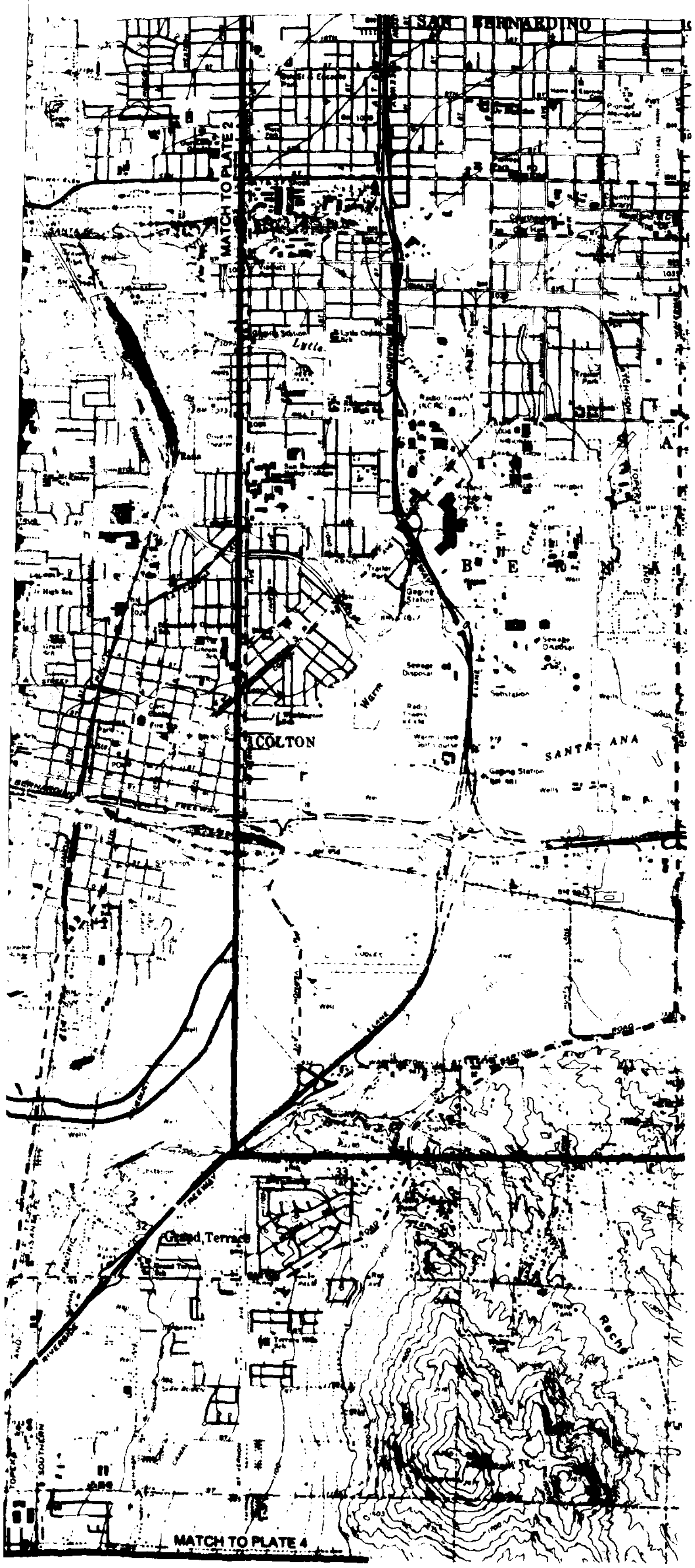
SEVEN OAKS COFFERDAM EMERGENCY PLAN

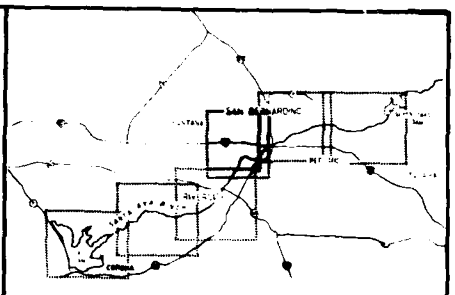
INUNDATION MAP

JUNE 1988

PLATE NO. 2


Cross Section - 23
Distance from Dam........... 16.7 Miles
Arrival Time..................... 5.0 Hours
Peak Elevation Time.......... 5.6 Hours
Peak Elevation.................. 868 Feet, NGVD
Average Overbank Depth.... 0 Feet
RIALTO
BLOOMINGTON
West Colton
CRESTMORE
SOUTHERN PACIFIC
SANTA ANA RIVER
Colton Rec Lake
Hermosa Cemetery
SAN BERNARDINO CO
RIVERSIDE CO
MATCH TO PLATE 2
0
23





VICINITY MAP

-N-

LEGEND

Limit of Flooded Area
With Dam Failure

Cofferdam

$\boxed{0}$ Average overbank depth in feet

26 ——— 26 Cross Section

2000 0 2000 4000 6000

SCALE IN FEET

National Geodetic vertical datum of 1929.
Source of base map: U.S. Geological Survey 7.5 minute series.
Contour interval 20 feet.
San Bernardino South, 1967, photo revised 1980
Fontana, 1967, photo revised 1980

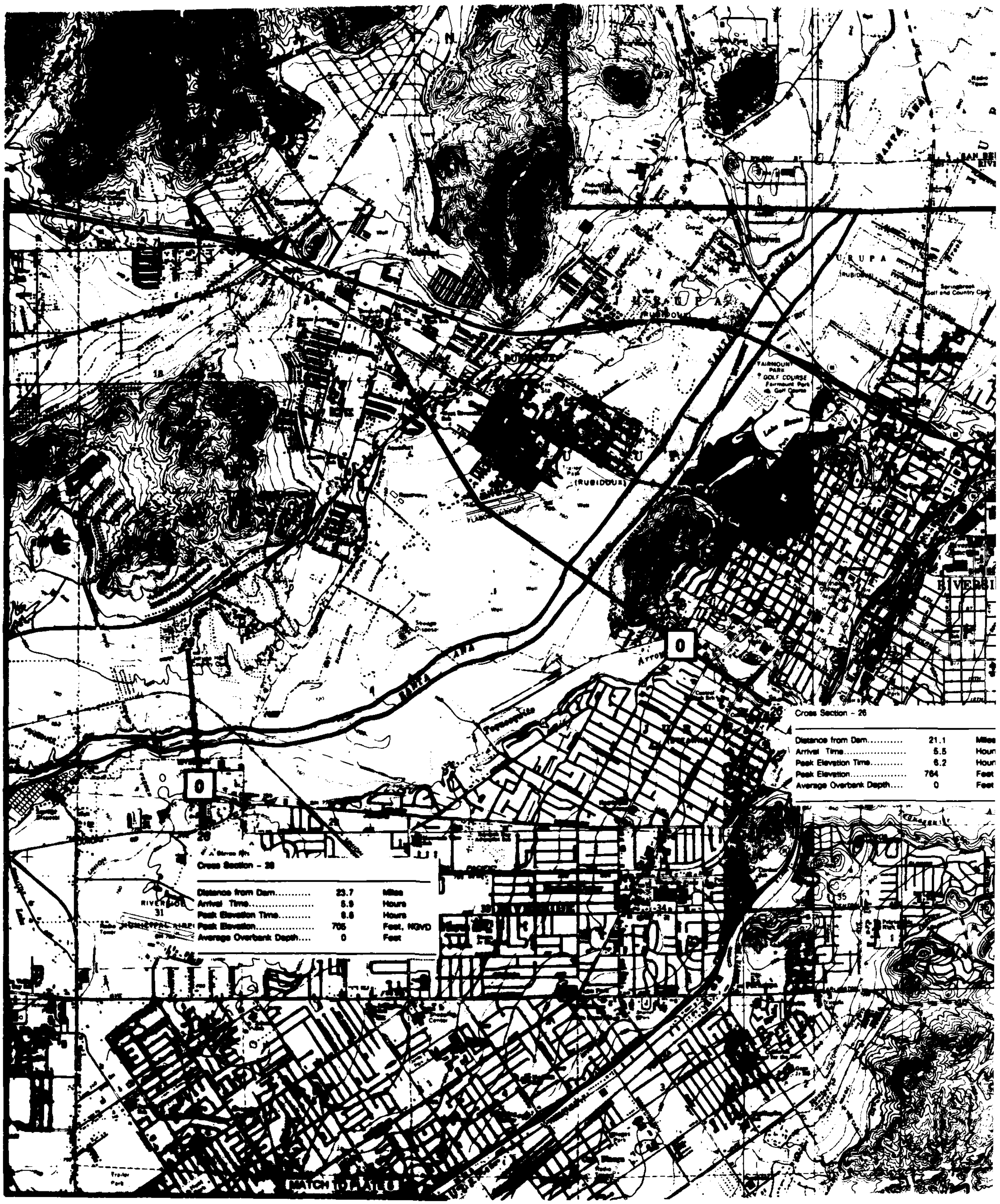
NOTE: The inundated areas shown on this map reflect potential flooding in the event of a cofferdam failure during construction of Seven Oaks Dam. The cofferdam was assumed to be 50 feet high.

U.S. ARMY ENGINEER DISTRICT, LOS ANGELES
CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

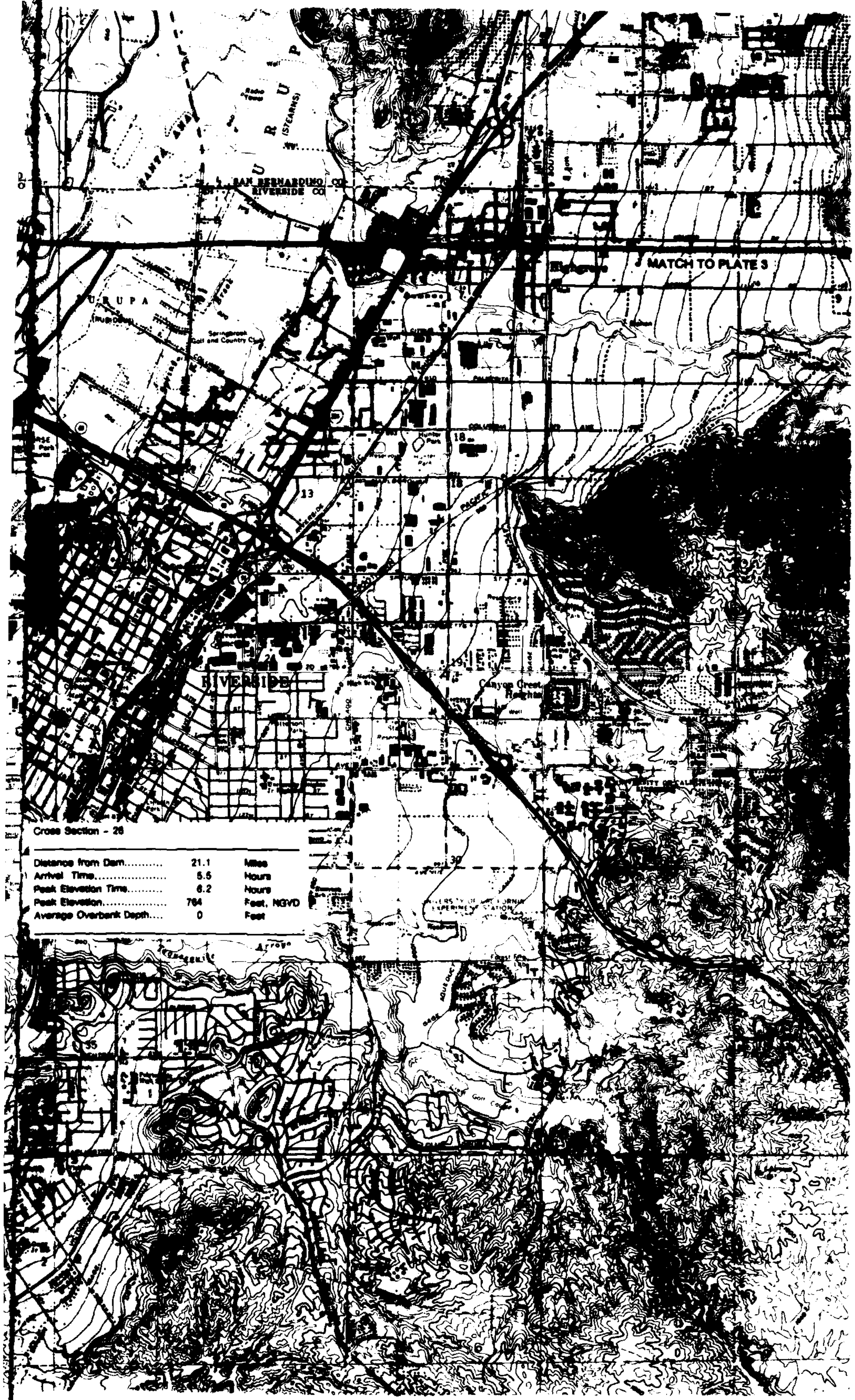
SEVEN OAKS COFFERDAM EMERGENCY PLAN

INUNDATION MAP

JUNE 1986 PLATE NO. 3

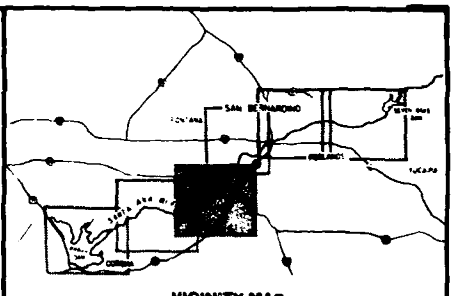


Cross Section - 26
Distance from Dam.......... 21.1 Miles
Arrival Time.................... 5.5 Hours
Peak Elevation Time.......... 6.2 Hours
Peak Elevation.................. 764 Feet
Average Overbank Depth.... 0 Feet
Cross Section -
Distance from Dam.......... 23.7 Miles
Arrival Time.................... Hours
Peak Elevation Time.......... Hours
Peak Elevation.................. 705 Feet, NGVD
Average Overbank Depth.... 0 Feet
FAIRMOUNT PARK GOLF COURSE
(RUBIDOUX)
RIVERSIDE
MATCH TO PLATE 6



Cross Section - 26

Distance from Dam	21.1	Miles
Arrival Time	5.5	Hours
Peak Elevation Time	6.2	Hours
Peak Elevation	764	Feet, NGVD
Average Overbank Depth	0	Feet



VICINITY MAP

N

LEGEND

Limit of Flooded Area With Dam Failure

Cofferdam

0 Average overbank depth in feet

26 — 26 Cross Section

2000 0 2000 4000 6000

SCALE IN FEET

National Geodetic vertical datum of 1929.
Source of base map: U.S. Geological Survey 7.5 minute series.
Contour interval 20 feet.
San Bernardino South, 1967, photo revised 1980
Fontana, 1967, photo revised 1980
Riverside East, 1967, photo revised 1980
Riverside West, 1967, photo revised 1980

NOTE: The inundated areas shown on this map reflect potential flooding in the event of a cofferdam failure during construction of Seven Oaks Dam. The cofferdam was assumed to be 50 feet high.

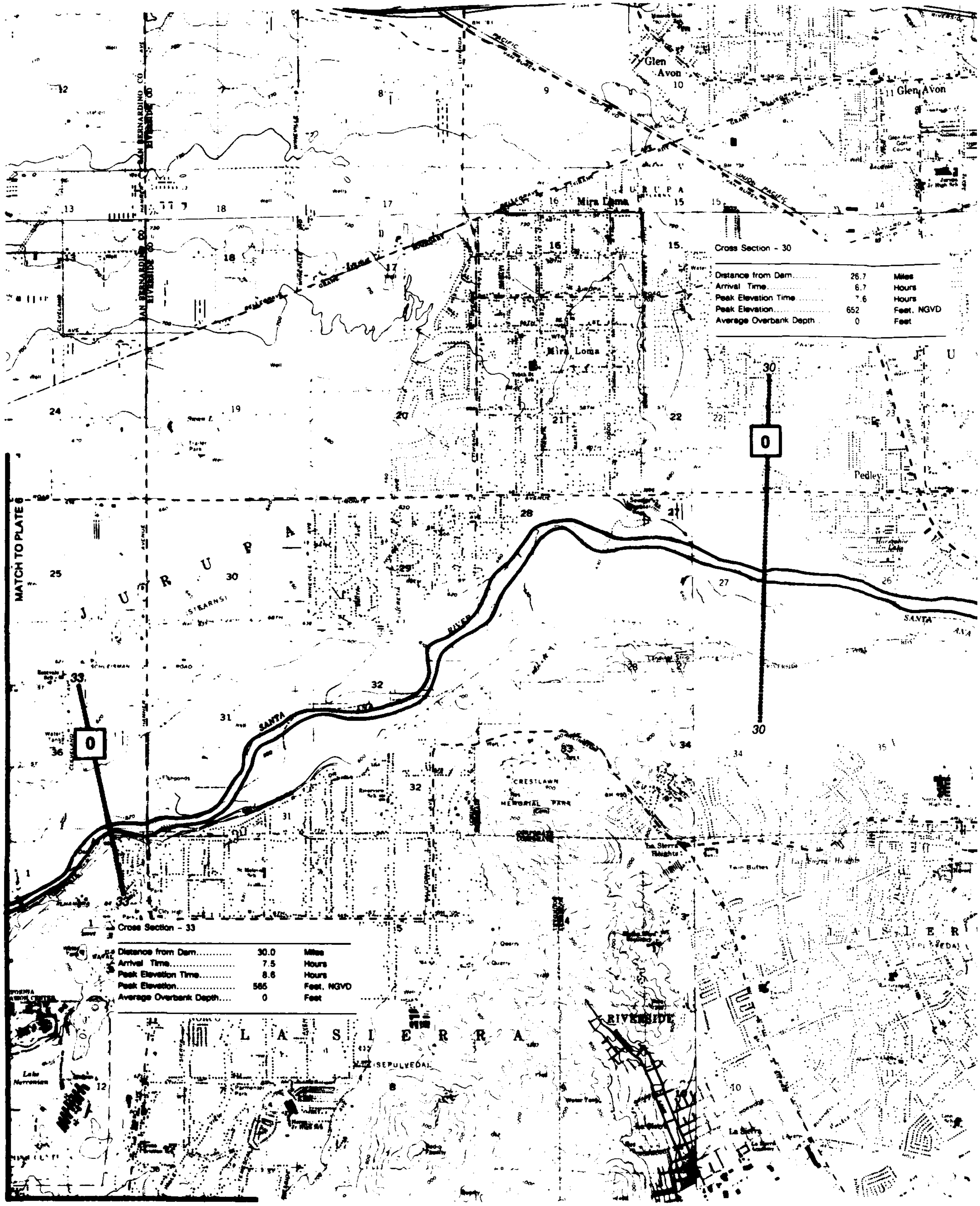
U.S. ARMY ENGINEER DISTRICT, LOS ANGELES
CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

SEVEN OAKS COFFERDAM EMERGENCY PLAN

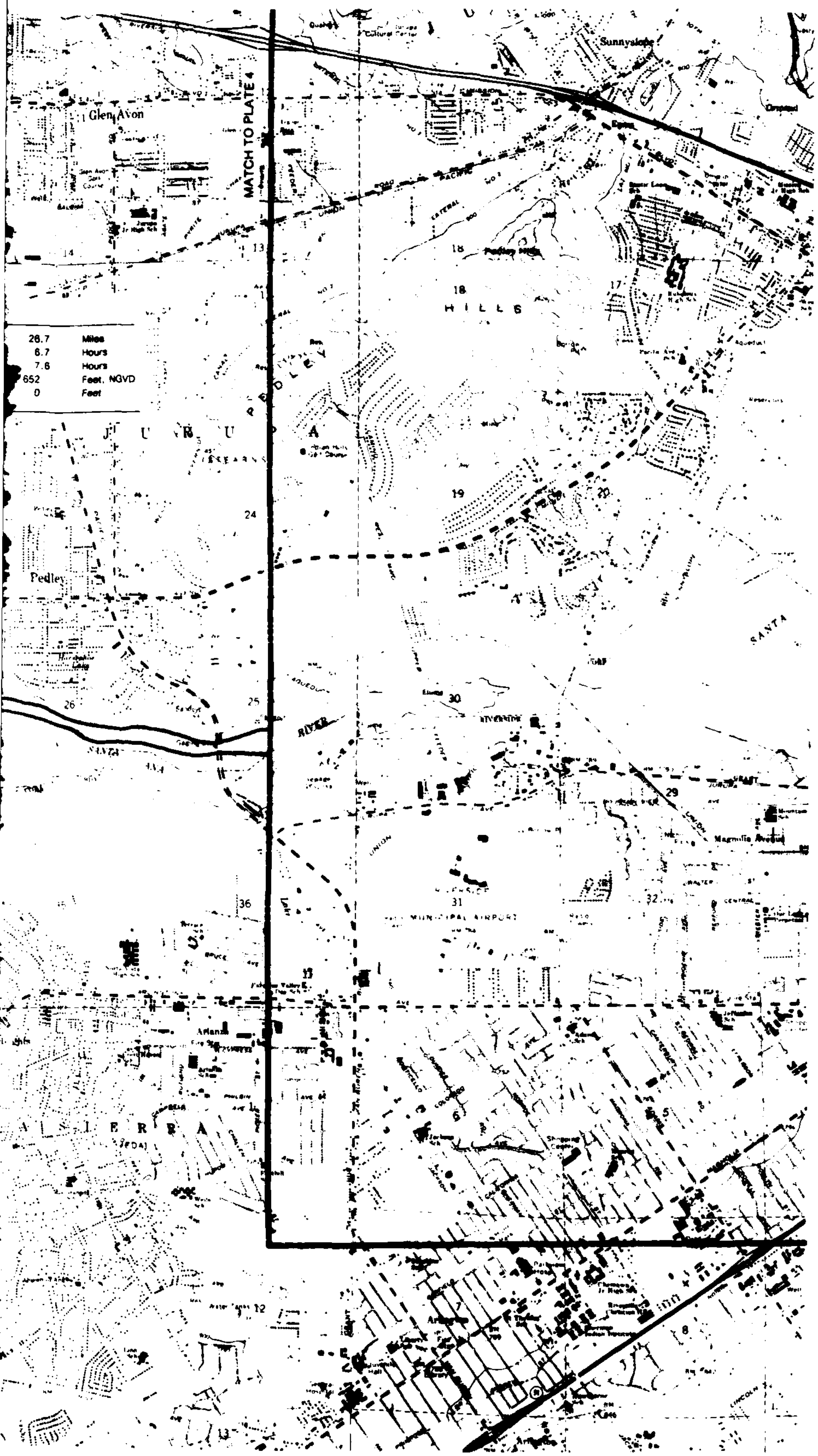
INUNDATION MAP

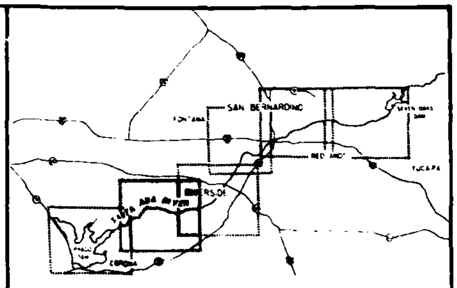
JUNE 1988

PLATE NO. 4



MATCH TO PLATE 6
Cross Section - 30
Distance from Dam 26.7 Miles
Arrival Time 6.7 Hours
Peak Elevation Time 7.6 Hours
Peak Elevation 652 Feet, NGVD
Average Overbank Depth 0 Feet
Cross Section - 33
Distance from Dam 30.0 Miles
Arrival Time 7.5 Hours
Peak Elevation Time 8.6 Hours
Peak Elevation 585 Feet, NGVD
Average Overbank Depth 0 Feet
Glen Avon
Mira Loma
Pedley
J U R U P A
L A S I E R R A
SANTA ANA RIVER
CRESTLAWN MEMORIAL PARK
La Sierra Heights
Twin Buttes
RIVERSIDE
La Sierra
Lake Norconian
SAN BERNARDINO CO.
RIVERSIDE CO.





VICINITY MAP

-N-

LEGEND

Limit of Flooded Area With Dam Failure

Cofferdam

$\boxed{0}$ Average overbank depth in feet

26 ——— 26 Cross Section

2000 0 2000 4000 6000

SCALE IN FEET

National Geodetic vertical datum of 1929.
Source of base map: U.S. Geological Survey 7.5 minute series.
Contour interval 10 feet.
Guasti, 1967, photo revised 1980
Contour interval 20 feet.
Fontana, 1967, photo revised 1980
Riverside West, 1967, photo revised 1980
Corona North, 1967, photo revised 1980

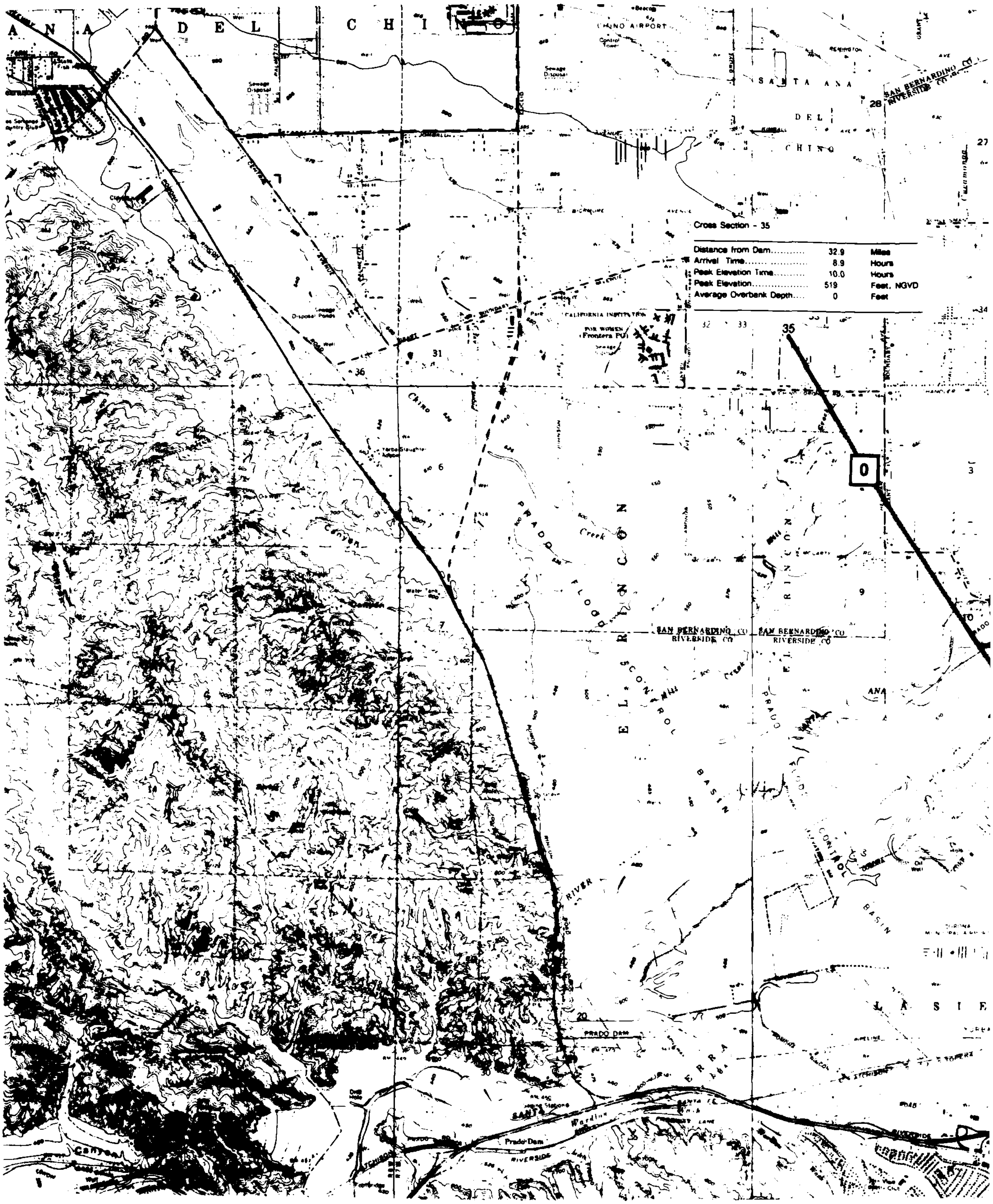
NOTE: The inundated areas shown on this map reflect potential flooding in the event of a cofferdam failure during construction of Seven Oaks Dam. The cofferdam was assumed to be 50 feet high.

U.S. ARMY ENGINEER DISTRICT, LOS ANGELES
CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

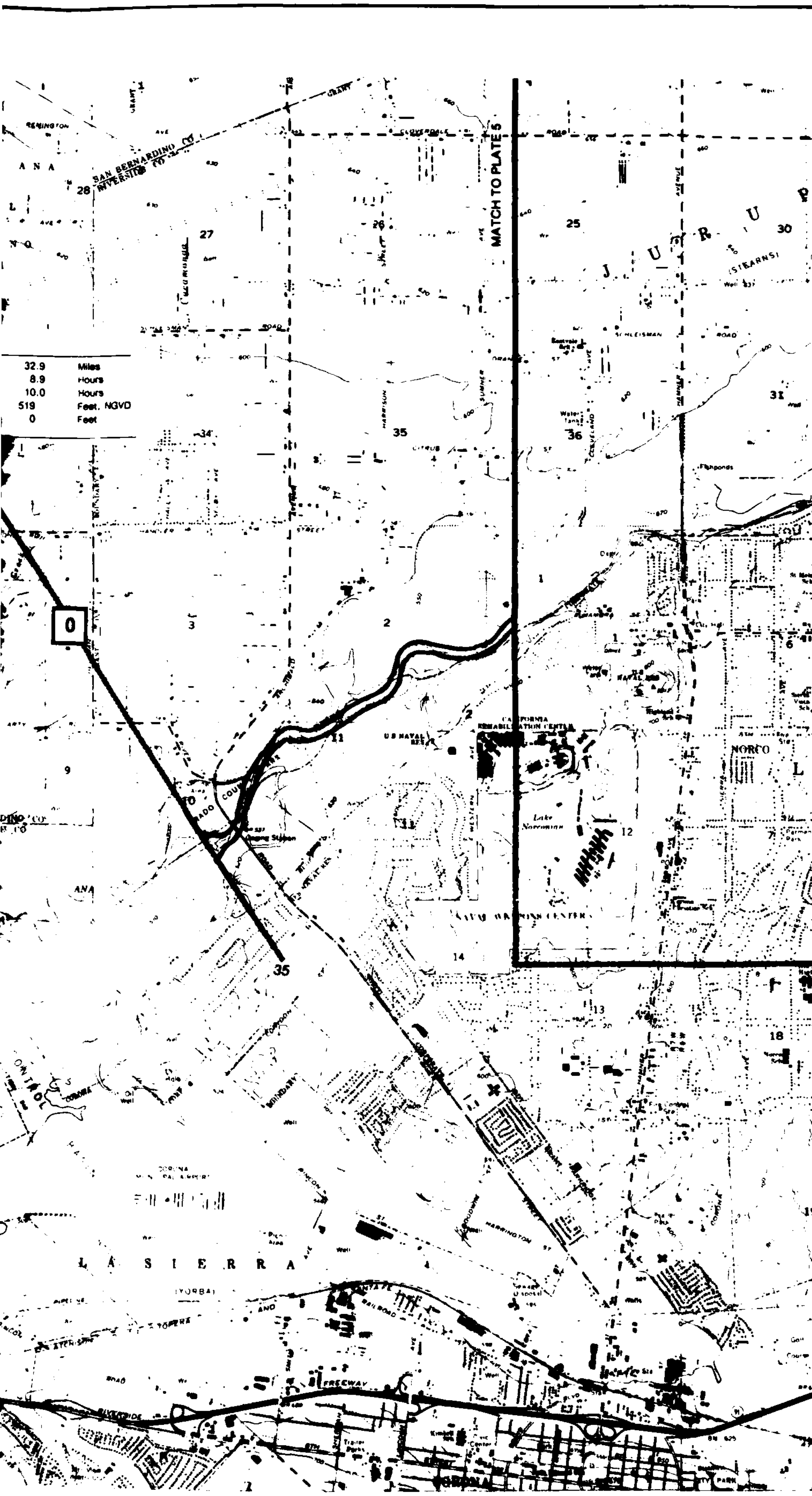
SEVEN OAKS COFFERDAM EMERGENCY PLAN

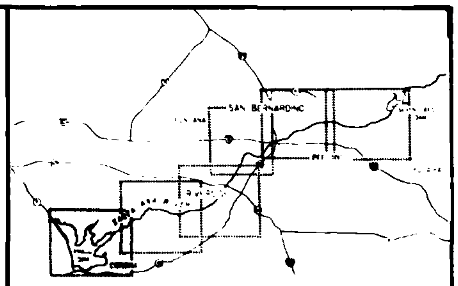
INUNDATION MAP

JUNE 1988 PLATE NO. 5



Cross Section - 35
Distance from Dam.......... 32.9 Miles
Arrival Time.......... 8.9 Hours
Peak Elevation Time.......... 10.0 Hours
Peak Elevation.......... 519 Feet, NGVD
Average Overbank Depth.... 0 Feet
35
0
CHINO AIRPORT
SANTA ANA DEL CHINO
CALIFORNIA INSTITUTION FOR WOMEN (Frontera PO)
SAN BERNARDINO CO
RIVERSIDE CO
PRADO FLOOD CONTROL BASIN
Chino Creek
Canyon
RIVER
PRADO Dam
Prado Dam
EL RINCON





VICINITY MAP

N

LEGEND

Cofferdam — Limit of Flooded Area With Dam Failure

0 Average overbank depth in feet

26 — 26 Cross Section

2000 0 2000 4000 6000

SCALE IN FEET

National Geodetic vertical datum of 1929.
Source of base map: U.S. Geological Survey 7.5 minute series.
Contour interval 25 feet.
Prado Dam, 1967, photo revised 1981
Contour interval 20 feet.
Corona North, 1967, photo revised 1980

NOTE: The inundated areas shown on this map reflect potential flooding in the event of a cofferdam failure during construction of Seven Oaks Dam. The cofferdam was assumed to be 50 feet high.

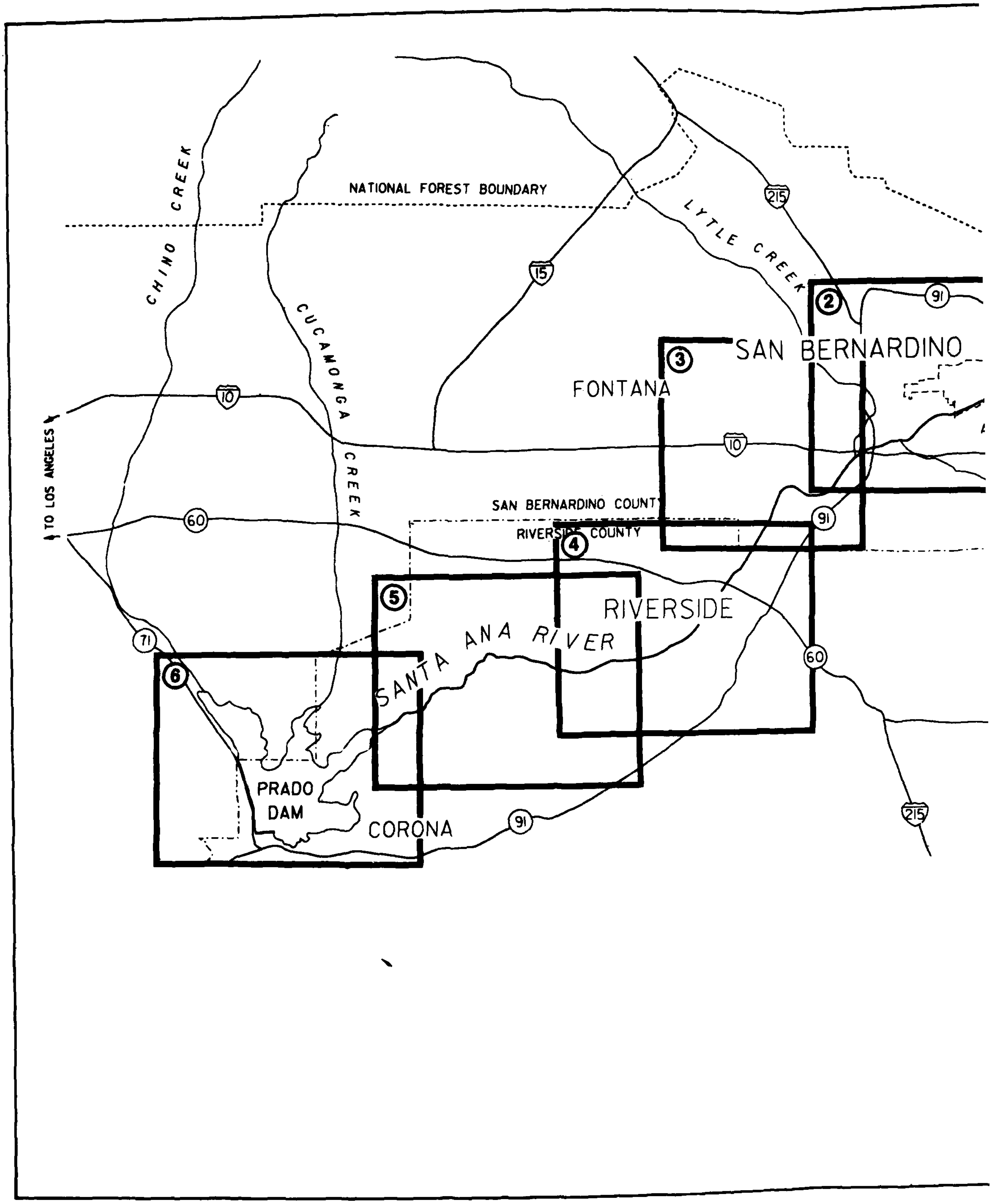
U.S. ARMY ENGINEER DISTRICT, LOS ANGELES
CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

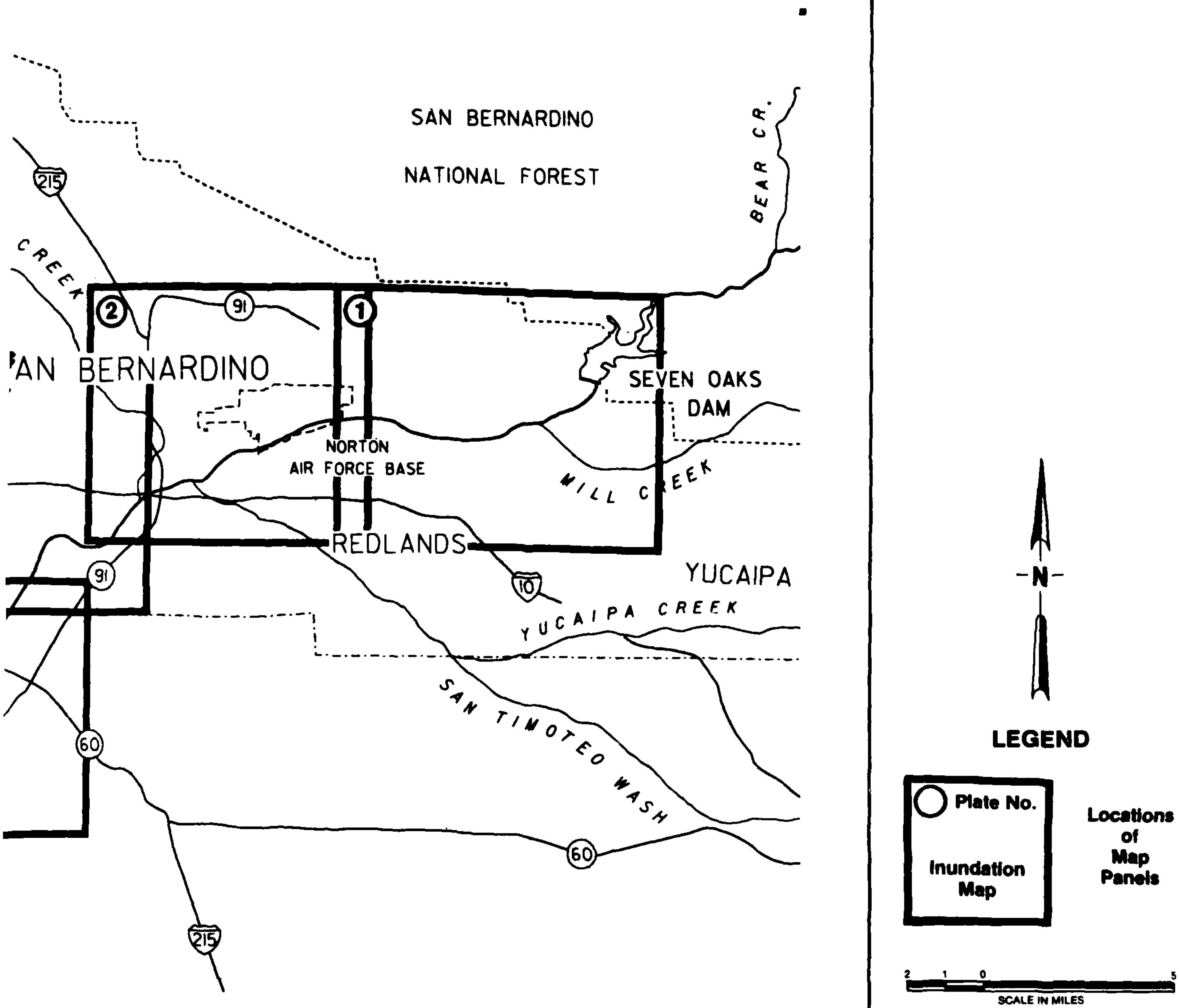
SEVEN OAKS COFFERDAM EMERGENCY PLAN

INUNDATION MAP

JUNE 1988

PLATE NO. 6


NATIONAL FOREST BOUNDARY
CHINO CREEK
CUCAMONGA CREEK
LYTLE CREEK
15
215
91
10
60
71
2
3
4
5
6
SAN BERNARDINO
FONTANA
TO LOS ANGELES
SAN BERNARDINO COUNTY
RIVERSIDE COUNTY
RIVERSIDE
SANTA ANA RIVER
PRADO DAM
CORONA



U.S. ARMY ENGINEER DISTRICT, LOS ANGELES
CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

SEVEN OAKS DAM
EMERGENCY PLAN

INDEX MAP

SEPTEMBER 1987

Construction Stage 100 Feet, Pool Elevation 2200 Feet, NGVD

Section Number	Plate Number	Distance from Dam (Miles)	Invert Elevation (Feet, NGVD)	Arrival Time (Hours)	Peak Elevation (Feet, NGVD)	Time of Peak Elevation (Hours)	Average Overbank Depth (Feet)
Dam	1	-	2100	-	N/A	0.2	-
5	1	0.7	1950	0.1	1973	0.2	13
10	1	3.8	1490	0.4	1503	0.7	3
13	1	7.0	1218	1.2	1227	1.4	1
15	2	8.8	1112	1.3	1121	1.8	0
20	2	12.9	959	2.2	968	2.9	0
23	3	16.7	861	2.9	870	3.4	1
26	4	21.1	758	3.4	766	4.0	0
28	4	23.7	698	3.9	707	4.4	1
30	5	26.7	646	4.4	653	4.9	1
33	5	30.0	559	5.2	567	5.7	1
35	6	32.9	514	5.8	521	6.4	3

Construction Stage 200 Feet, Pool Elevation 230(

Section Number	Plate Number	Distance from Dam (Miles)	Invert Elevation (Feet, NGVD)
Dam	1	-	2100
5	1	0.7	1950
10	1	3.8	1490
13	1	7.0	1218
15	2	8.8	1112
20	2	12.9	959
23	3	16.7	861
26	4	21.1	758
28	4	23.7	698
30	5	26.7	646
33	5	30.0	559
35	6	32.9	514

Construction Stage 300 Feet, Pool Elevation 2400 Feet, NGVD

Section Number	Plate Number	Distance from Dam (Miles)	Invert Elevation (Feet, NGVD)	Arrival Time (Hours)	Peak Elevation (Feet, NGVD)	Time of Peak Elevation (Hours)	Average Overbank Depth (Feet)
Dam	1	-	2100	-	N/A	0.4	-
5	1	0.7	1950	0.1	2009	0.4	49
10	1	3.8	1490	0.3	1519	0.6	19
13	1	7.0	1218	0.8	1246	1.0	20
15	2	8.8	1112	1.0	1134	1.3	8
20	2	12.9	959	1.7	982	2.9	4
23	3	16.7	861	2.2	881	3.3	11
26	4	21.1	758	2.7	777	4.0	17
28	4	23.7	698	2.9	721	4.5	11
30	5	26.7	646	3.3	662	4.7	9
33	5	30.0	559	3.8	579	5.2	10
35	6	32.9	514	4.1	532	5.5	9

Construction Stage 400 Feet, Pool Elevation 2500

Section Number	Plate Number	Distance from Dam (Miles)	Invert Elevation (Feet, NGVD)
Dam	1	-	2100
5	1	0.7	1950
10	1	3.8	1490
13	1	7.0	1218
15	2	8.8	1112
20	2	12.9	959
23	3	16.7	861
26	4	21.1	758
28	4	23.7	698
30	5	26.7	646
33	5	30.0	559
35	6	32.9	514

Full Height Dam with Failure at Spillway Crest, 2598.0 Feet NGVD

Section Number	Plate Number	Distance from Dam (Miles)	Invert Elevation (Feet, NGVD)	Arrival Time (Hours)	Peak Elevation (Feet, NGVD)	Time of Peak Elevation (Hours)	Average Overbank Depth (Feet)
Dam	1	-	2100	-	N/A	0.5	-
5	1	0.7	1950	0.1	2026	0.5	66
10	1	3.8	1490	0.3	1534	0.6	34
13	1	7.0	1218	0.6	1262	0.8	36
15	2	8.8	1112	0.8	1152	1.1	23
20	2	12.9	959	1.5	1008	2.1	24
23	3	16.7	861	1.9	898	2.6	28
26	4	21.1	758	2.3	790	3.3	30
28	4	23.7	698	2.8	741	3.7	31
30	5	26.7	646	3.1	675	3.9	21
33	5	30.0	559	3.4	594	4.2	24
35	6	32.9	514	3.9	546	4.5	20

Construction Stage 200 Feet, Pool Elevation 2300 Feet, NGVD

Section Number	Plate Number	Distance from Dam (Miles)	Invert Elevation (Feet, NGVD)	Arrival Time (Hours)	Peak Elevation (Feet, NGVD)	Time of Peak Elevation (Hours)	Average Overbank Depth (Feet)
Dam	1	–	2100	–	N/A	0.4	–
5	1	0.7	1950	0.1	1994	0.4	34
10	1	3.8	1490	0.4	1512	0.6	12
13	1	7.0	1218	1.1	1239	1.4	15
15	2	8.8	1112	1.3	1127	1.6	3
20	2	12.9	959	1.9	976	3.0	0
23	3	16.7	861	2.5	876	3.4	6
26	4	21.1	758	3.0	772	3.8	0
28	4	23.7	698	3.4	715	4.1	5
30	5	26.7	646	3.8	658	4.5	6
33	5	30.0	559	4.4	575	5.2	6
35	6	32.9	514	4.9	527	5.7	6

Construction Stage 400 Feet, Pool Elevation 2500 Feet, NGVD

Section Number	Plate Number	Distance from Dam (Miles)	Invert Elevation (Feet, NGVD)	Arrival Time (Hours)	Peak Elevation (Feet, NGVD)	Time of Peak Elevation (Hours)	Average Overbank Depth (Feet)
Dam	1	–	2100	–	N/A	0.5	–
5	1	0.7	1950	0.1	2014	0.5	54
10	1	3.8	1490	0.3	1527	0.6	27
13	1	7.0	1218	0.6	1253	0.9	27
15	2	8.8	1112	0.8	1143	1.2	14
20	2	12.9	959	1.6	1000	2.6	16
23	3	16.7	861	2.2	889	3.1	19
26	4	21.1	758	2.5	784	3.9	24
28	4	23.7	698	2.8	731	4.3	21
30	5	26.7	646	3.2	668	4.6	14
33	5	30.0	559	3.8	586	4.9	16
35	6	32.9	514	4.3	539	5.2	13

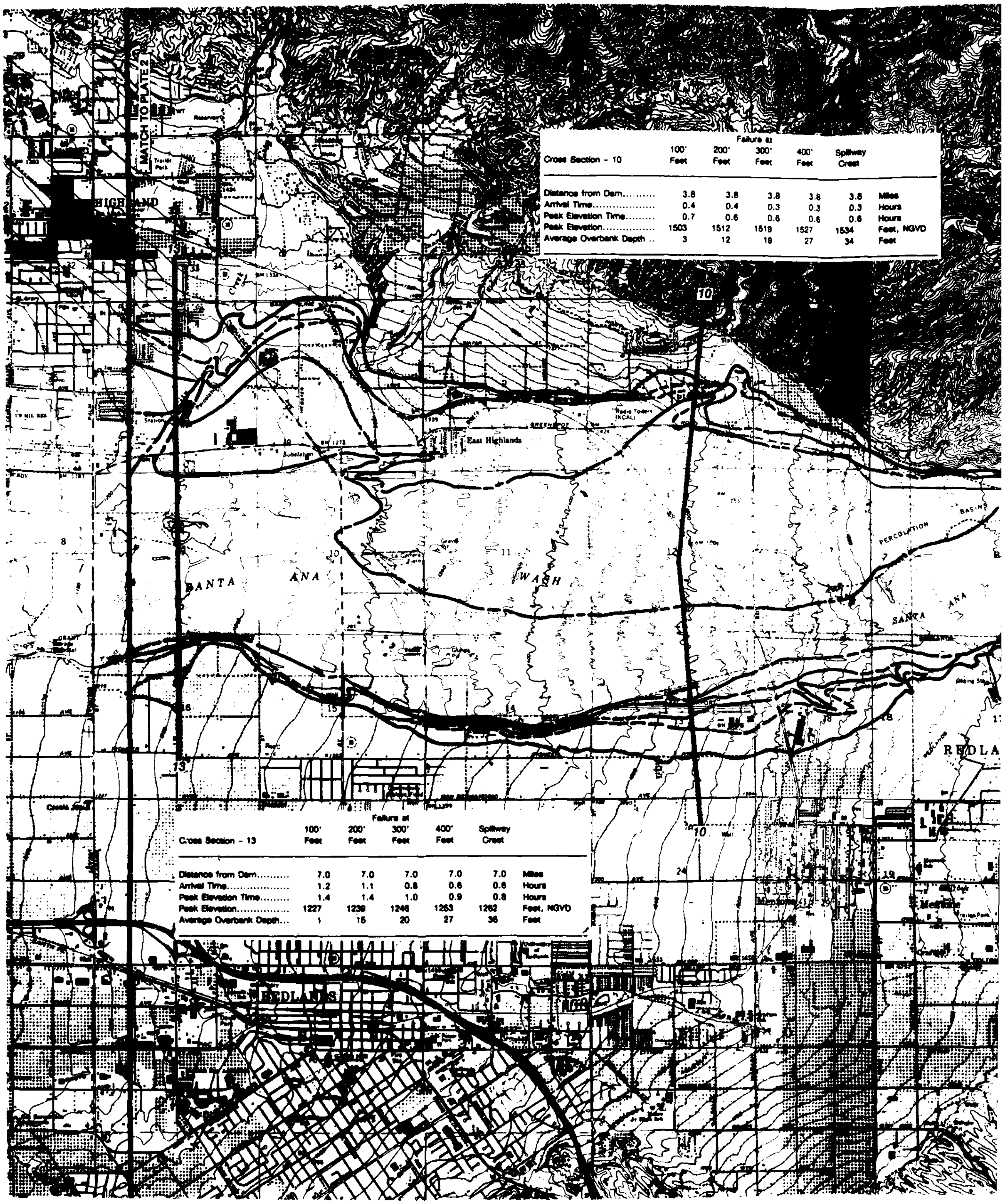
n D)	Time of Peak Elevation (Hours)	Average Overbank Depth (Feet)
	0.5	–
	0.5	66
	0.6	34
	0.8	36
	1.1	23
	2.1	24
	2.6	28
	3.3	30
	3.7	31
	3.9	21
	4.2	24
	4.5	20

U.S. ARMY ENGINEER DISTRICT, LOS ANGELES
CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

SEVEN OAKS DAM EMERGENCY PLAN

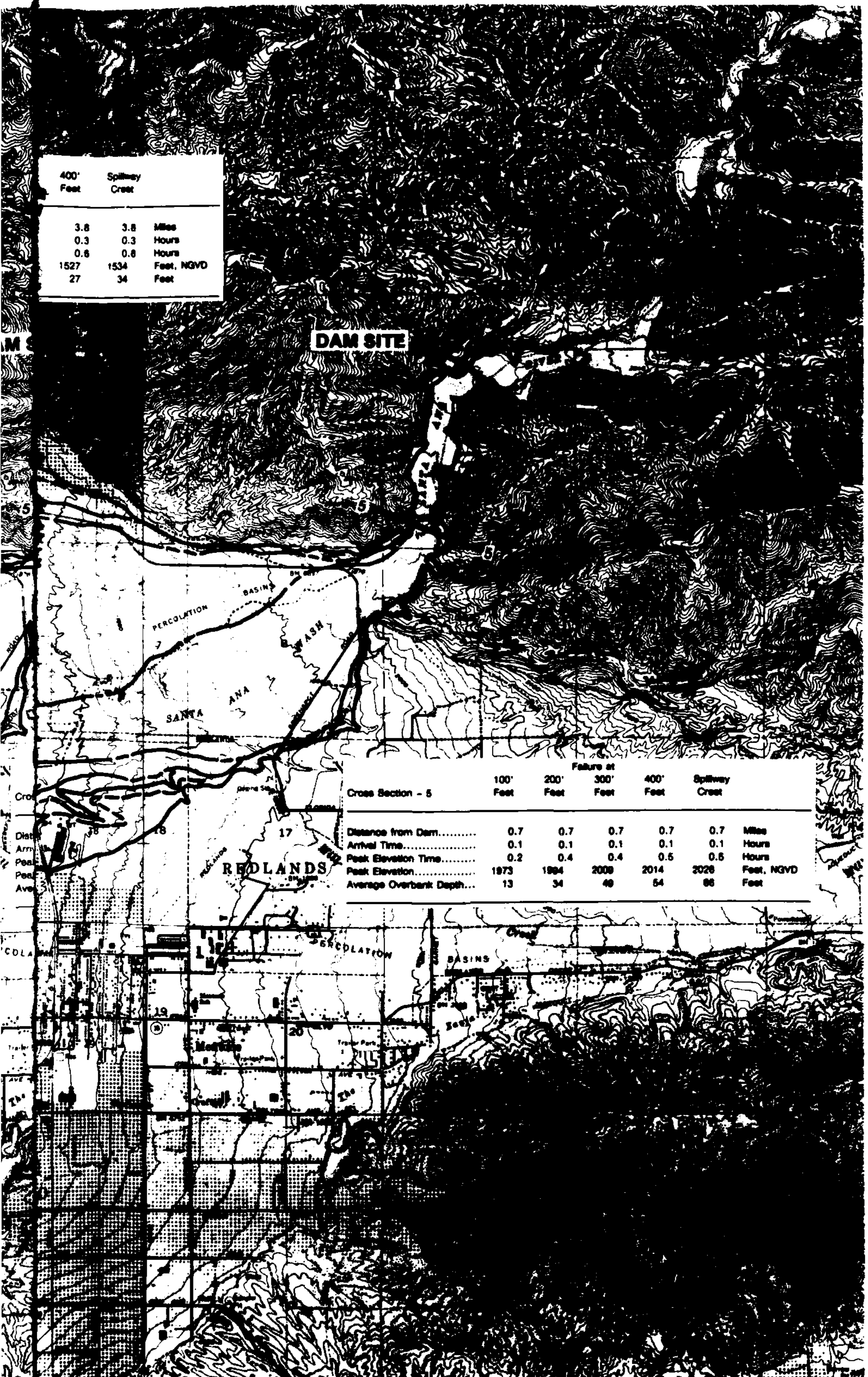
INDEX

SEPTEMBER 1987



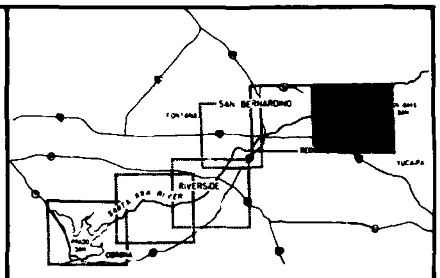
Cross Section - 10	Failure at 100' Feet	200' Feet	300' Feet	400' Feet	Spillway Crest	
Distance from Dam..........	3.8	3.8	3.8	3.8	3.8	Miles
Arrival Time....................	0.4	0.4	0.3	0.3	0.3	Hours
Peak Elevation Time.........	0.7	0.6	0.6	0.6	0.6	Hours
Peak Elevation..............	1503	1512	1519	1527	1534	Feet, NGVD
Average Overbank Depth ..	3	12	19	27	34	Feet

Cross Section - 13	Failure at 100' Feet	200' Feet	300' Feet	400' Feet	Spillway Crest	
Distance from Dam..........	7.0	7.0	7.0	7.0	7.0	Miles
Arrival Time....................	1.2	1.1	0.8	0.6	0.6	Hours
Peak Elevation Time.........	1.4	1.4	1.0	0.9	0.8	Hours
Peak Elevation..............	1227	1239	1246	1253	1262	Feet, NGVD
Average Overbank Depth...	1	15	20	27	36	Feet

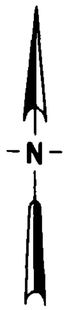


400' Feet	Spillway Crest	
3.8	3.8	Miles
0.3	0.3	Hours
0.6	0.6	Hours
1527	1534	Feet, NGVD
27	34	Feet

Cross Section - 5	Failure at 100' Feet	200' Feet	300' Feet	400' Feet	Spillway Crest	
Distance from Dam..........	0.7	0.7	0.7	0.7	0.7	Miles
Arrival Time...................	0.1	0.1	0.1	0.1	0.1	Hours
Peak Elevation Time.........	0.2	0.4	0.4	0.5	0.6	Hours
Peak Elevation................	1973	1994	2009	2014	2026	Feet, NGVD
Average Overbank Depth....	13	34	49	54	66	Feet



VICINITY MAP



LEGEND

Limit of Flood Area With Dam Failure

100 feet

200 feet

300 feet

400 feet

Spillway Crest (498')

26 ——— 26 Cross Section

2000 0 2000 4000 6000

SCALE IN FEET

National Geodetic vertical datum of 1929.
Source of base map: U.S. Geological Survey 7.5 minute series.
Contour interval 20 feet.
Redlands, 1967, photo revised 1980
Contour interval 40 feet.
Harrison Mtn., 1967, photo revised 1980
Keller Peak, 1967
Yucaipa, 1967, photo revised 1980

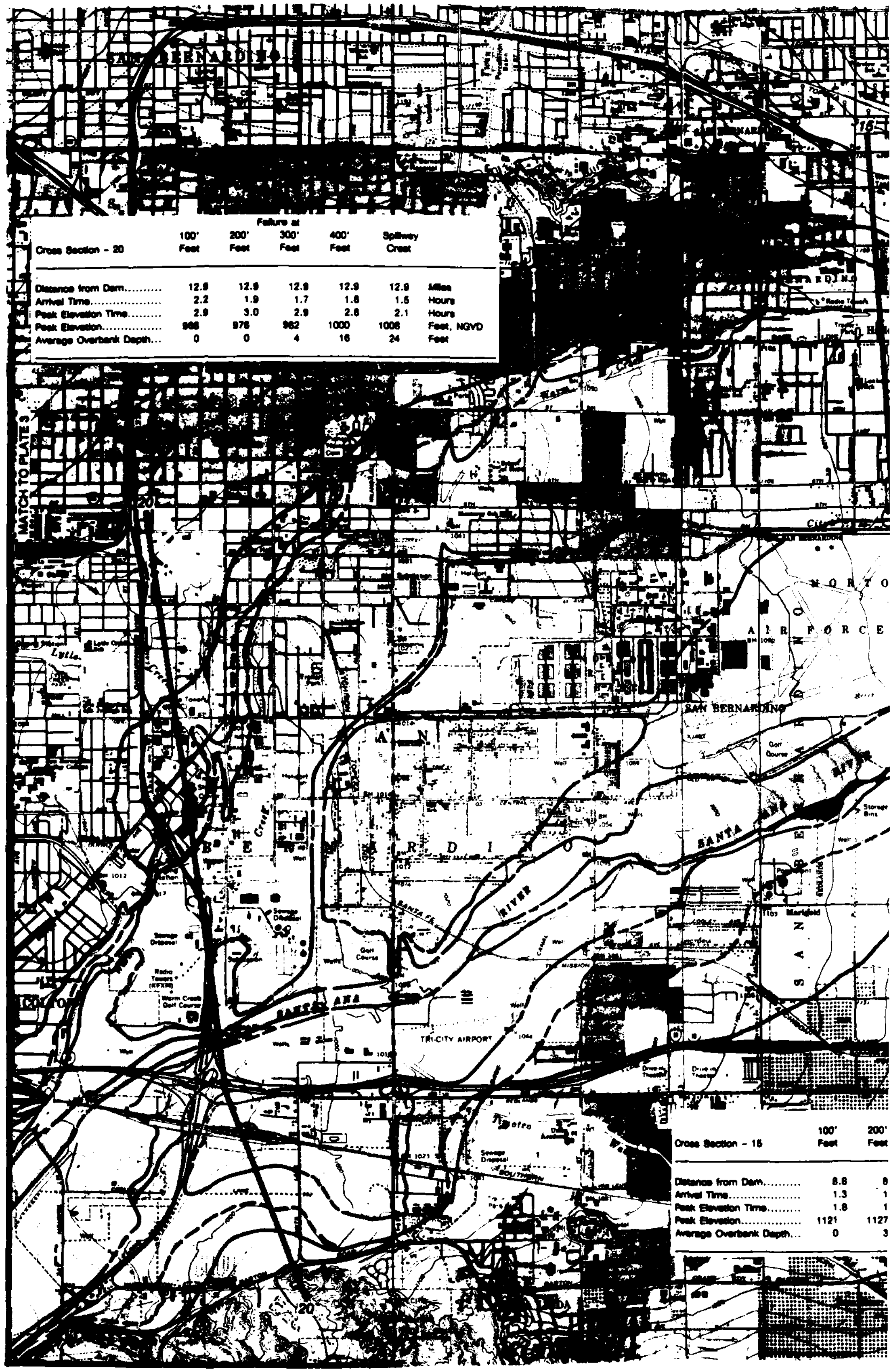
NOTE: The inundated areas shown on this map reflect potential flooding in the event of a dam failure during construction of Seven Oaks Dam. The flooded area boundaries are for progressive construction stages as shown in the legend and in the cross-section information boxes.

U.S. ARMY ENGINEER DISTRICT, LOS ANGELES
CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

SEVEN OAKS DAM
EMERGENCY PLAN

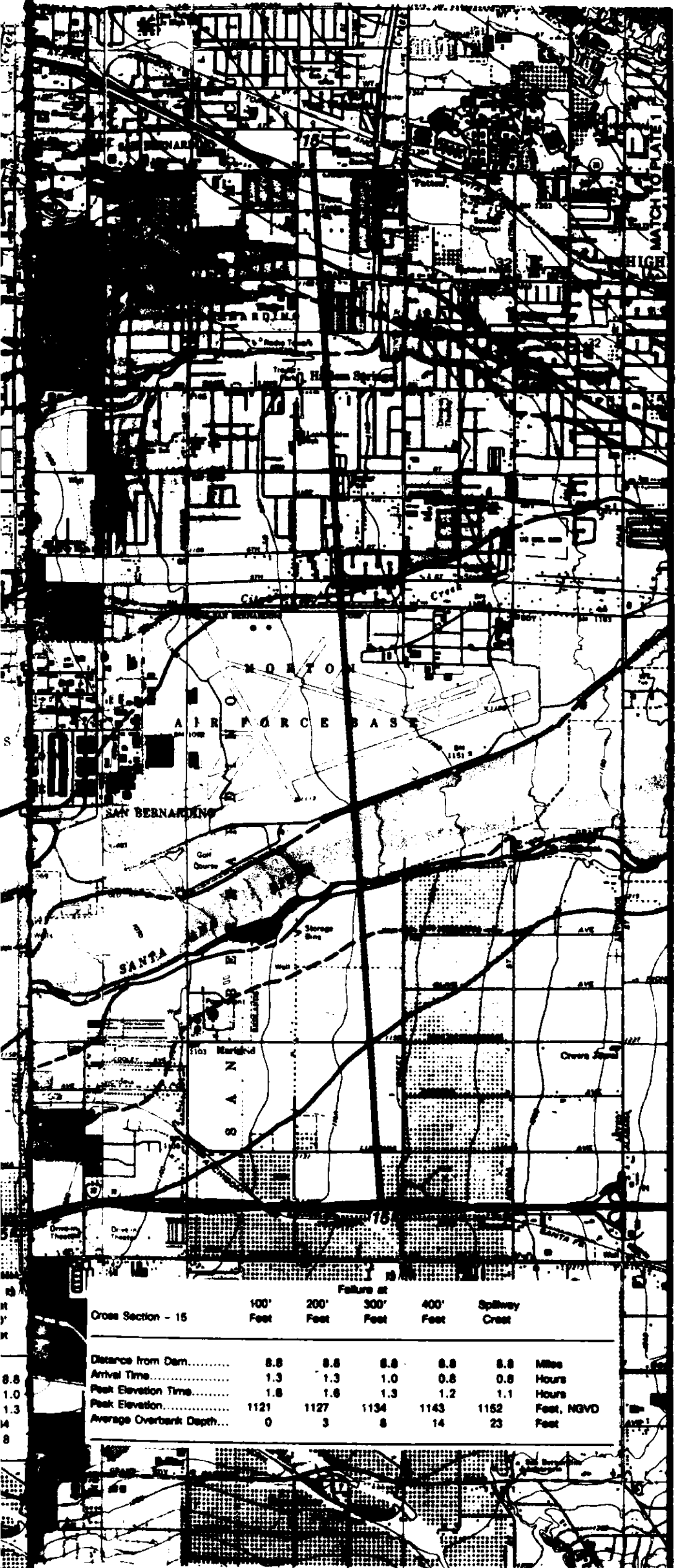
INUNDATION MAP

SEPTEMBER 1987 PLATE NO. 1

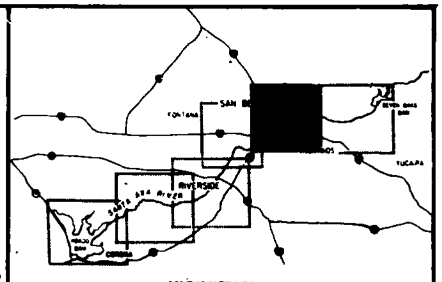


Cross Section - 20	Failure at 100' Feet	200' Feet	300' Feet	400' Feet	Spillway Crest	
Distance from Dam..........	12.9	12.9	12.9	12.9	12.9	Miles
Arrival Time....................	2.2	1.9	1.7	1.6	1.5	Hours
Peak Elevation Time.........	2.9	3.0	2.9	2.6	2.1	Hours
Peak Elevation.................	968	976	982	1000	1006	Feet, NGVD
Average Overbank Depth...	0	0	4	16	24	Feet

Cross Section - 15	100' Feet	200' Feet
Distance from Dam..........	8.6	8
Arrival Time....................	1.3	1
Peak Elevation Time.........	1.8	1
Peak Elevation.................	1121	1127
Average Overbank Depth...	0	3



Cross Section - 15	Failure at 100' Feet	200' Feet	300' Feet	400' Feet	Spillway Crest	
Distance from Dam..........	8.8	8.8	8.8	8.8	8.8	Miles
Arrival Time..................	1.3	1.3	1.0	0.8	0.8	Hours
Peak Elevation Time.........	1.6	1.6	1.3	1.2	1.1	Hours
Peak Elevation.................	1121	1127	1134	1143	1152	Feet, NGVD
Average Overbank Depth...	0	3	8	14	23	Feet



VICINITY MAP

-N-

LEGEND

Limit of Flood Area With Dam Failure

100 feet
200 feet
300 feet
400 feet
Spillway Crest (496')

26 — 26 Cross Section

2000 0 2000 4000 6000

SCALE IN FEET

National Geodetic vertical datum of 1929.
Source of base map: U.S. Geological Survey 7.5 minute series.
Contour interval 20 feet.
Redlands, 1967, photo revised 1980
San Bernardino South, 1967, photo revised 1980
Contour interval 40 feet.
San Bernardino North, 1967, photo revised 1980
Harrison Mtn., 1967, photo revised 1980

NOTE: The inundated areas shown on this map reflect potential flooding in the event of a dam failure during construction of Seven Oaks Dam. The flooded area boundaries are for progressive construction stages as shown in the legend and in the cross-section information boxes.

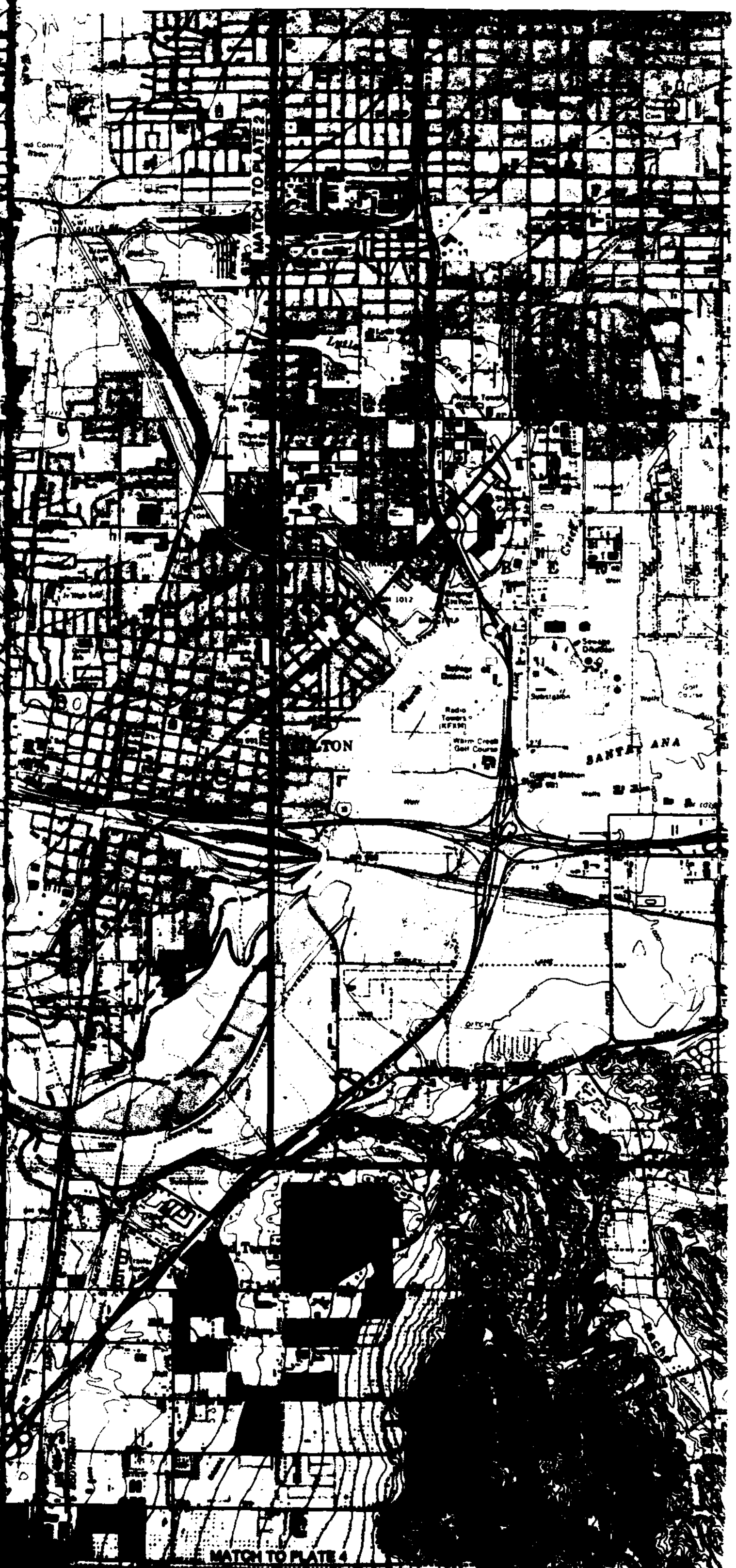
U.S. ARMY ENGINEER DISTRICT, LOS ANGELES
CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

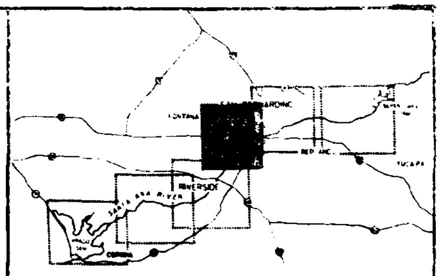
SEVEN OAKS DAM
EMERGENCY PLAN

INUNDATION MAP

SEPTEMBER 1987 PLATE NO. 2

Cross Section - 23	Failure at 100' Feet	200' Feet	300' Feet	400' Feet	Spillway Crest	
Distance from Dam..........	16.7	16.7	16.7	16.7	16.7	Miles
Arrival Time....................	2.9	2.5	2.2	2.2	1.9	Hours
Peak Elevation Time.........	3.4	3.4	3.3	3.1	2.8	Hours
Peak Elevation................	870	876	881	889	896	Feet, NGVD
Average Overbank Depth...	0	6	11	19	28	Feet





VICINITY MAP

-N-

LEGEND

Limit of Flood Area With Dam Failure

100 feet
200 feet
300 feet
400 feet
Spillway Crest (496')

26 — 26 Cross Section

2000 0 2000 4000 6000

SCALE IN FEET

National Geodetic vertical datum of 1929.
Source of base map: U.S. Geological Survey 7.5 minute series.
Contour interval 20 feet.
San Bernardino South, 1967, photo revised 1980
Fontana, 1967, photo revised 1980

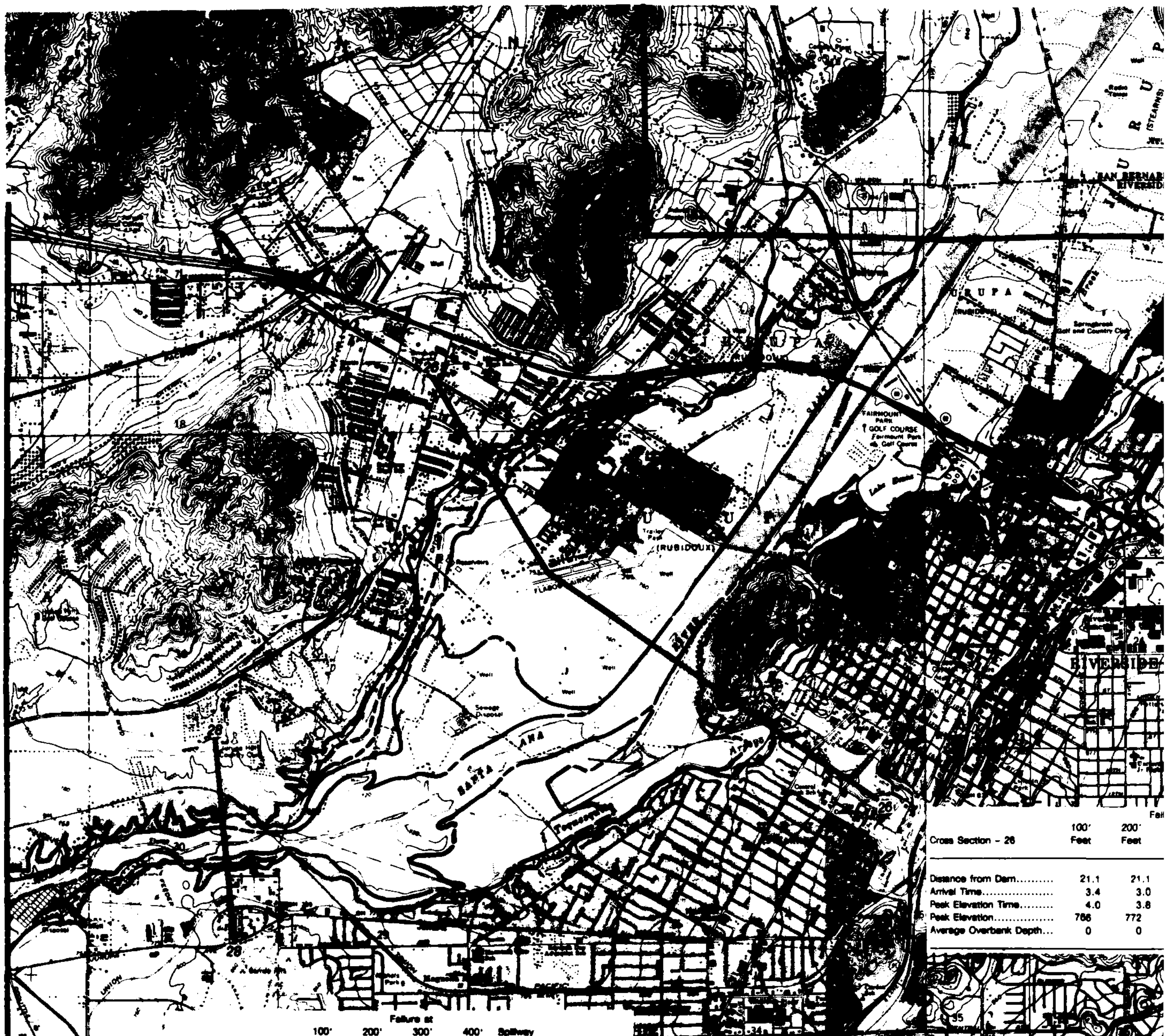
NOTE: The inundated areas shown on this map reflect potential flooding in the event of a dam failure during construction of Seven Oaks Dam. The flooded area boundaries are for progressive construction stages as shown in the legend and in the cross-section information boxes.

U.S. ARMY ENGINEER DISTRICT, LOS ANGELES
CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

SEVEN OAKS DAM EMERGENCY PLAN

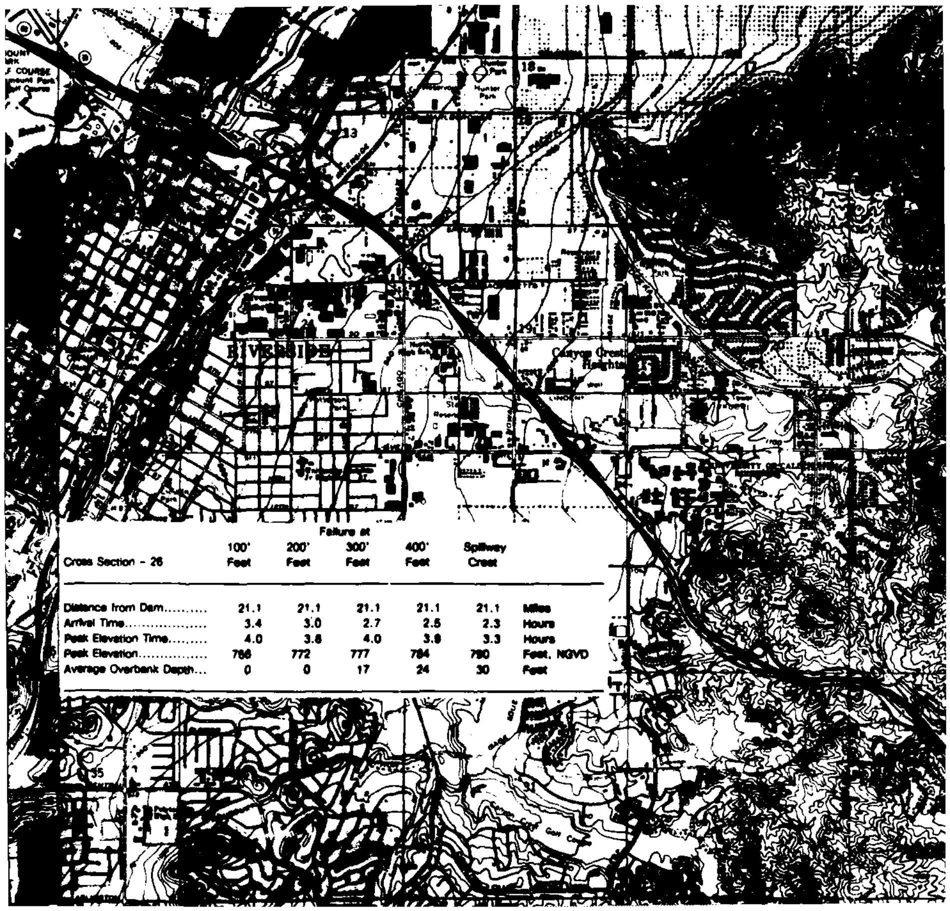
INUNDATION MAP

SEPTEMBER 1987 PLATE NO. 3

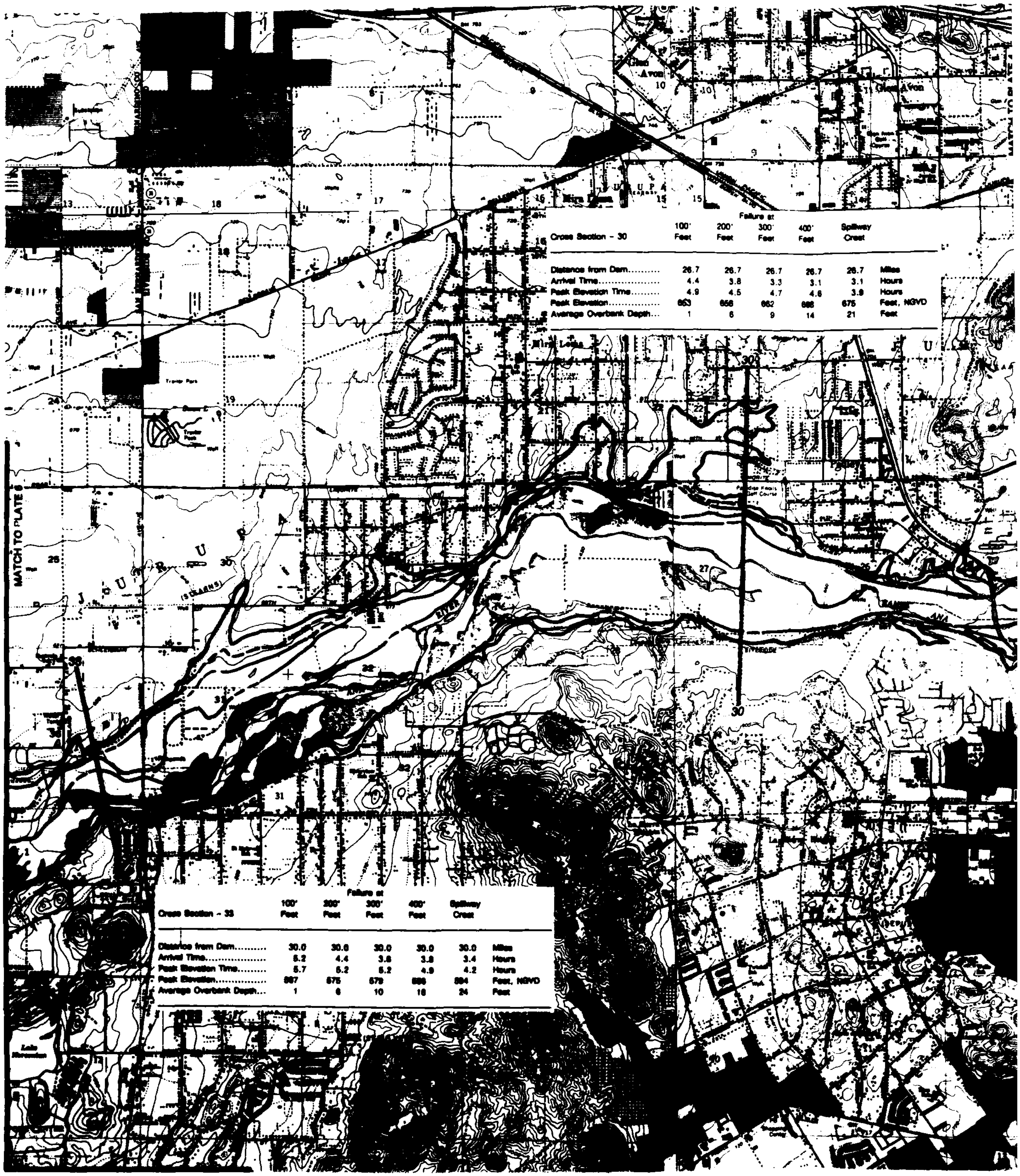


Cross Section - 28	Failure at 100' Feet	200' Feet	300' Feet	400' Feet	Spillway Crest	
Distance from Dam..........	23.7	23.7	23.7	23.7	23.7	Miles
Arrival Time....................	3.9	3.4	2.9	2.8	2.8	Hours
Peak Elevation Time.........	4.4	4.1	4.5	4.3	3.7	Hours
Peak Elevation.................	707	715	721	731	741	Feet, NGVD
Average Overbank Depth...	1	5	11	21	31	Feet

Cross Section - 26	Fai[lure at] 100' Feet	200' Feet
Distance from Dam..........	21.1	21.1
Arrival Time....................	3.4	3.0
Peak Elevation Time.........	4.0	3.8
Peak Elevation.................	766	772
Average Overbank Depth...	0	0



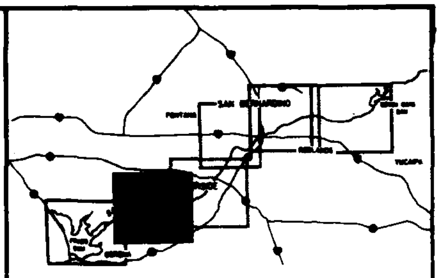
Cross Section - 26	Failure at 100' Feet	200' Feet	300' Feet	400' Feet	Spillway Crest	
Distance from Dam..........	21.1	21.1	21.1	21.1	21.1	Miles
Arrival Time....................	3.4	3.0	2.7	2.5	2.3	Hours
Peak Elevation Time.........	4.0	3.6	4.0	3.9	3.3	Hours
Peak Elevation................	766	772	777	784	790	Feet, NGVD
Average Overbank Depth...	0	0	17	24	30	Feet



Cross Section - 30	Failure at 100' Feet	200' Feet	300' Feet	400' Feet	Spillway Crest	
Distance from Dam..........	26.7	26.7	26.7	26.7	26.7	Miles
Arrival Time..................	4.4	3.8	3.3	3.1	3.1	Hours
Peak Elevation Time.........	4.9	4.5	4.7	4.8	3.9	Hours
Peak Elevation................	653	656	662	668	675	Feet, NGVD
Average Overbank Depth...	1	6	9	14	21	Feet

Cross Section - 33	Failure at 100' Feet	200' Feet	300' Feet	400' Feet	Spillway Crest	
Distance from Dam..........	30.0	30.0	30.0	30.0	30.0	Miles
Arrival Time..................	5.2	4.4	3.8	3.8	3.4	Hours
Peak Elevation Time.........	5.7	5.2	5.2	4.9	4.2	Hours
Peak Elevation................	667	675	679	686	694	Feet, NGVD
Average Overbank Depth...	1	6	10	16	24	Feet

Failure at	300' Feet	400' Feet	Spillway Crest	
7	26.7	26.7	26.7	Miles
8	3.3	3.1	3.1	Hours
5	4.7	4.6	3.9	Hours
8	662	668	675	Feet, NGVD
6	9	14	21	Feet



VICINITY MAP

N

LEGEND

Limit of Flood Area With Dam Failure

100 feet

200 feet

300 feet

400 feet

Spillway Crest (496')

26 — 26 Cross Section

2000 0 2000 4000 6000

SCALE IN FEET

National Geodetic vertical datum of 1929.
Source of base map: U.S. Geological Survey 7.5 minute series.
Contour interval 10 feet.
Guasti, 1967, photo revised 1980
Contour interval 20 feet.
Fontana, 1967, photo revised 1980
Riverside West, 1967, photo revised 1980
Corona North, 1967, photo revised 1980

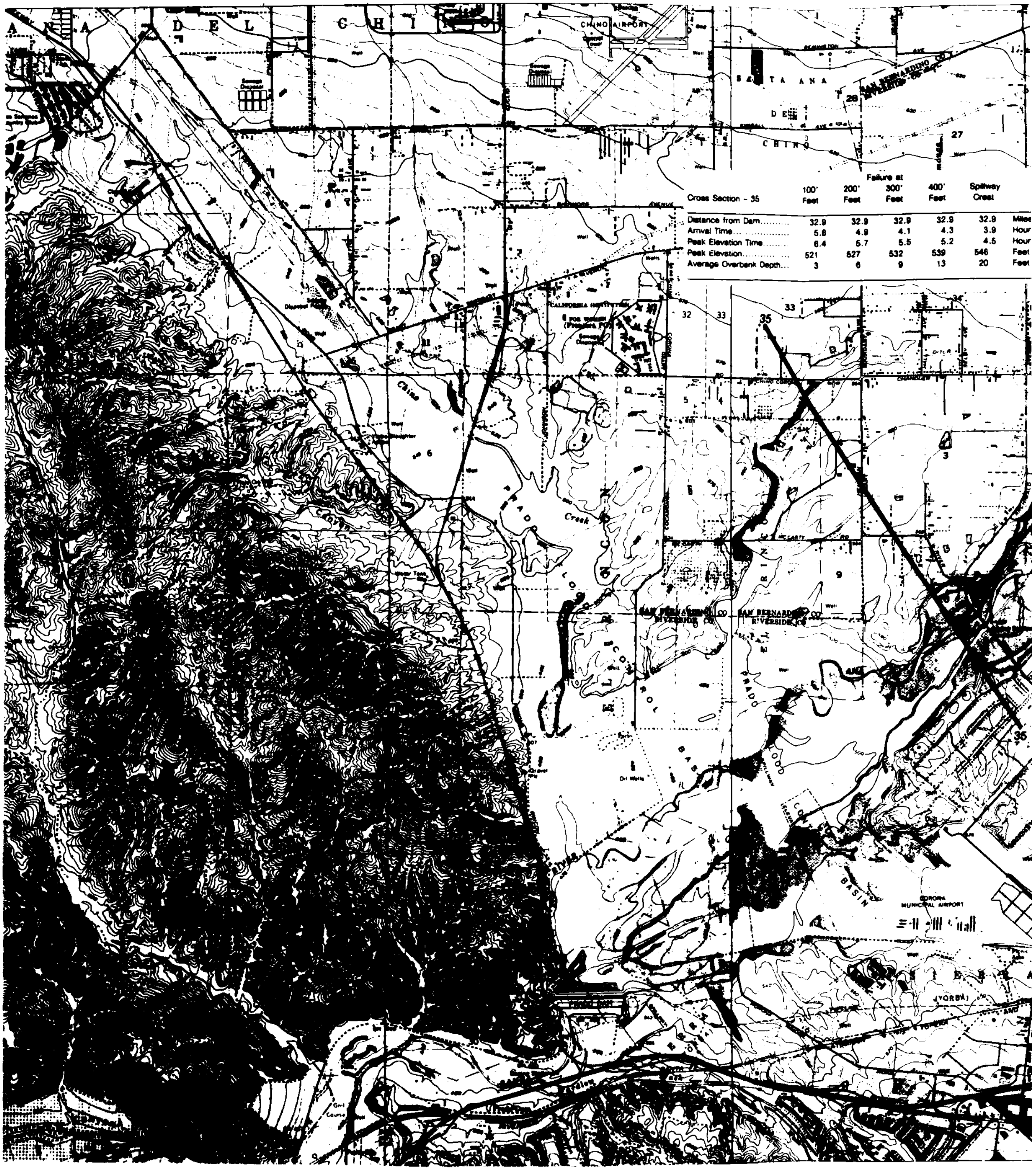
NOTE: The inundated areas shown on this map reflect potential flooding in the event of a dam failure during construction of Seven Oaks Dam. The flooded area boundaries are for progressive construction stages as shown in the legend and in the cross-section information boxes.

U.S. ARMY ENGINEER DISTRICT, LOS ANGELES
CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

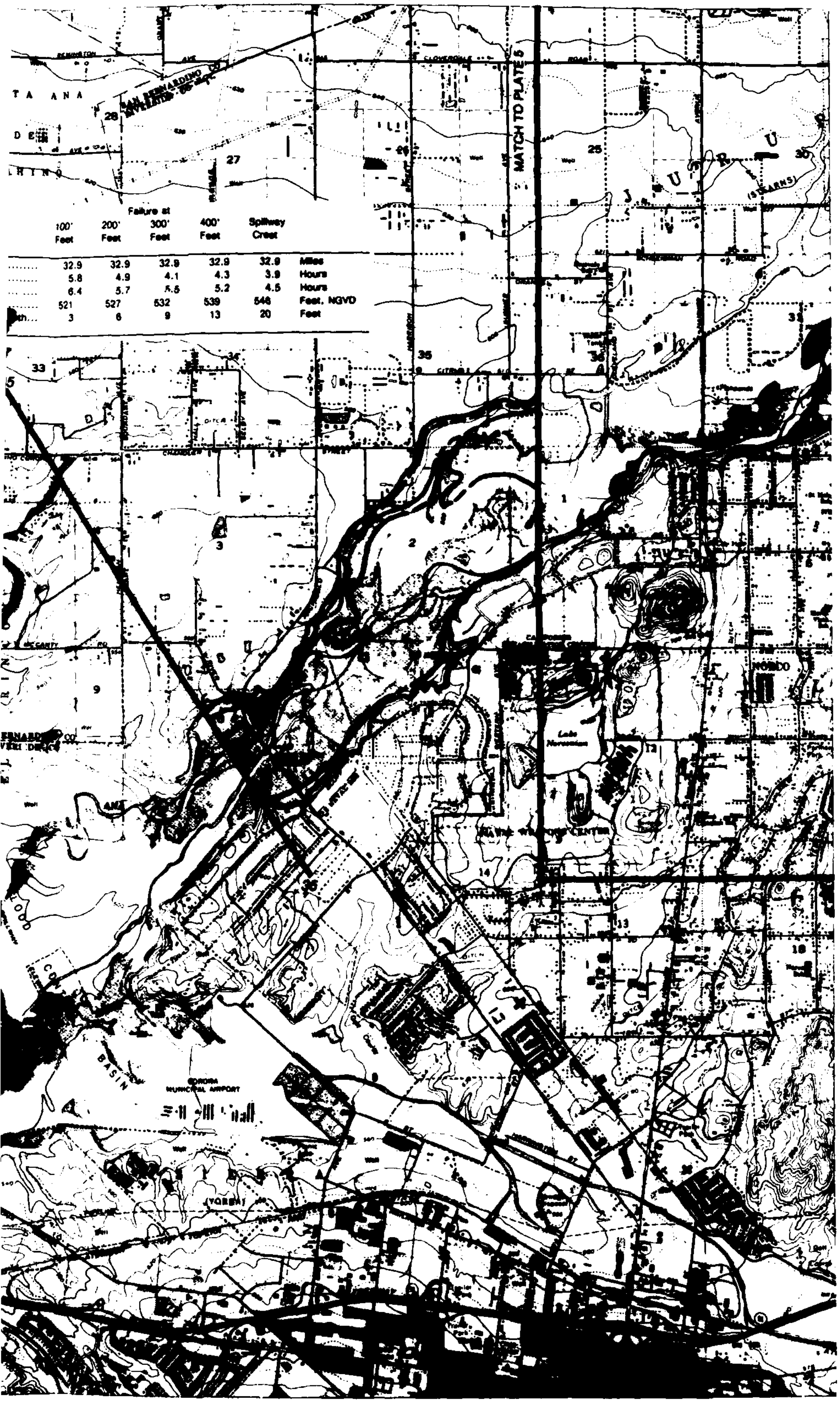
SEVEN OAKS DAM EMERGENCY PLAN

INUNDATION MAP

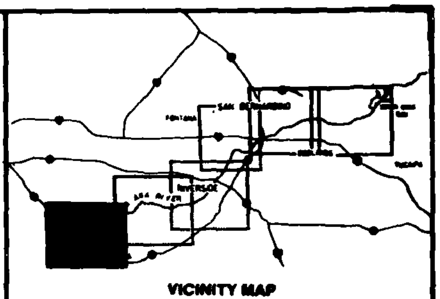
SEPTEMBER 1987 PLATE NO. 5



Cross Section - 35	Failure at 100' Feet	200' Feet	300' Feet	400' Feet	Spillway Crest	
Distance from Dam	32.9	32.9	32.9	32.9	32.9	Miles
Arrival Time	5.8	4.9	4.1	4.3	3.9	Hour
Peak Elevation Time	6.4	5.7	5.5	5.2	4.5	Hour
Peak Elevation	521	527	532	539	546	Feet
Average Overbank Depth	3	6	9	13	20	Feet



	Failure at 100' Feet	200' Feet	300' Feet	400' Feet	Spillway Crest	
....	32.9	32.9	32.9	32.9	32.9	Miles
....	5.8	4.9	4.1	4.3	3.9	Hours
....	6.4	5.7	5.5	5.2	4.5	Hours
....	521	527	532	539	546	Feet, NGVD
...th...	3	6	9	13	20	Feet



VICINITY MAP

-N-

LEGEND

Limit of Flood Area With Dam Failure

100 feet
200 feet
300 feet
400 feet
Spillway Crest (496')

26 —— 26 Cross Section

2000 0 2000 4000 6000

SCALE IN FEET

National Geodetic vertical datum of 1929.
Source of base map: U.S. Geological Survey 7.5 minute series.
Contour interval 25 feet.
Prado Dam, 1967, photo revised 1981
Contour interval 20 feet.
Corona North, 1967, photo revised 1980

NOTE: The inundated areas shown on this map reflect potential flooding in the event of a dam failure during construction of Seven Oaks Dam. The flooded area boundaries are for progressive construction stages as shown in the legend and in the cross-section information boxes.

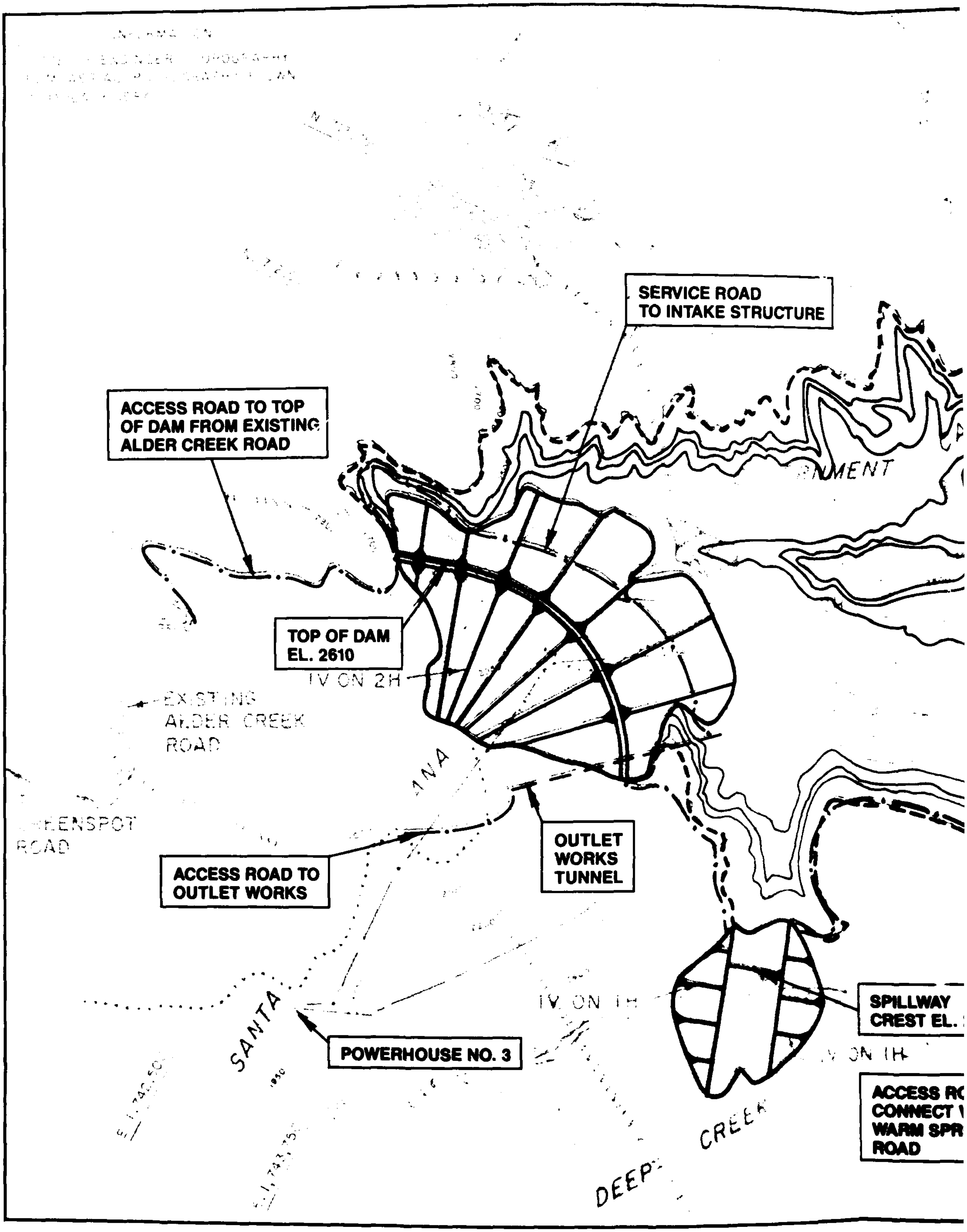
U.S. ARMY ENGINEER DISTRICT, LOS ANGELES
CORPS OF ENGINEERS
LOS ANGELES, CALIFORNIA

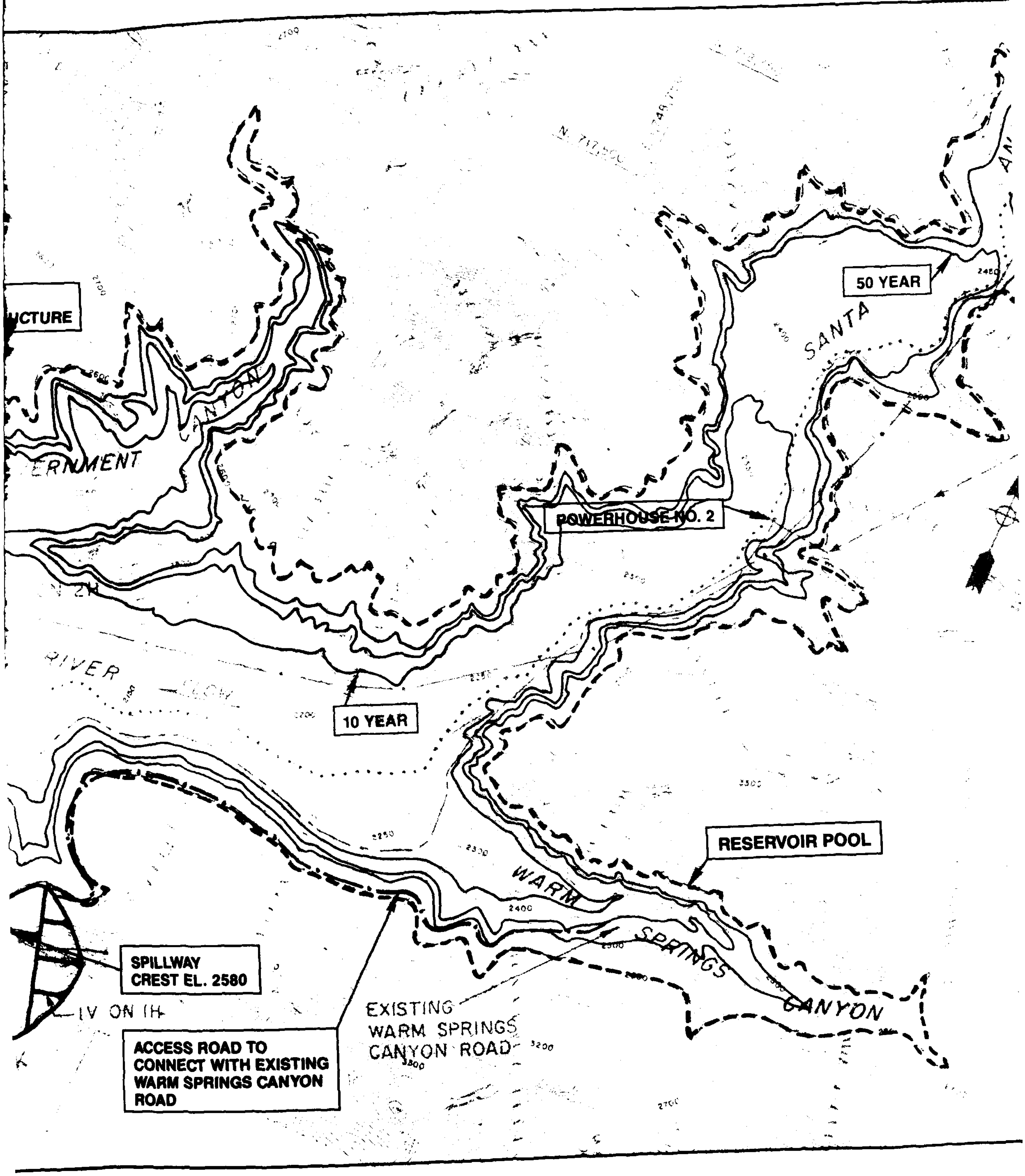
SEVEN OAKS DAM
EMERGENCY PLAN

INUNDATION MAP

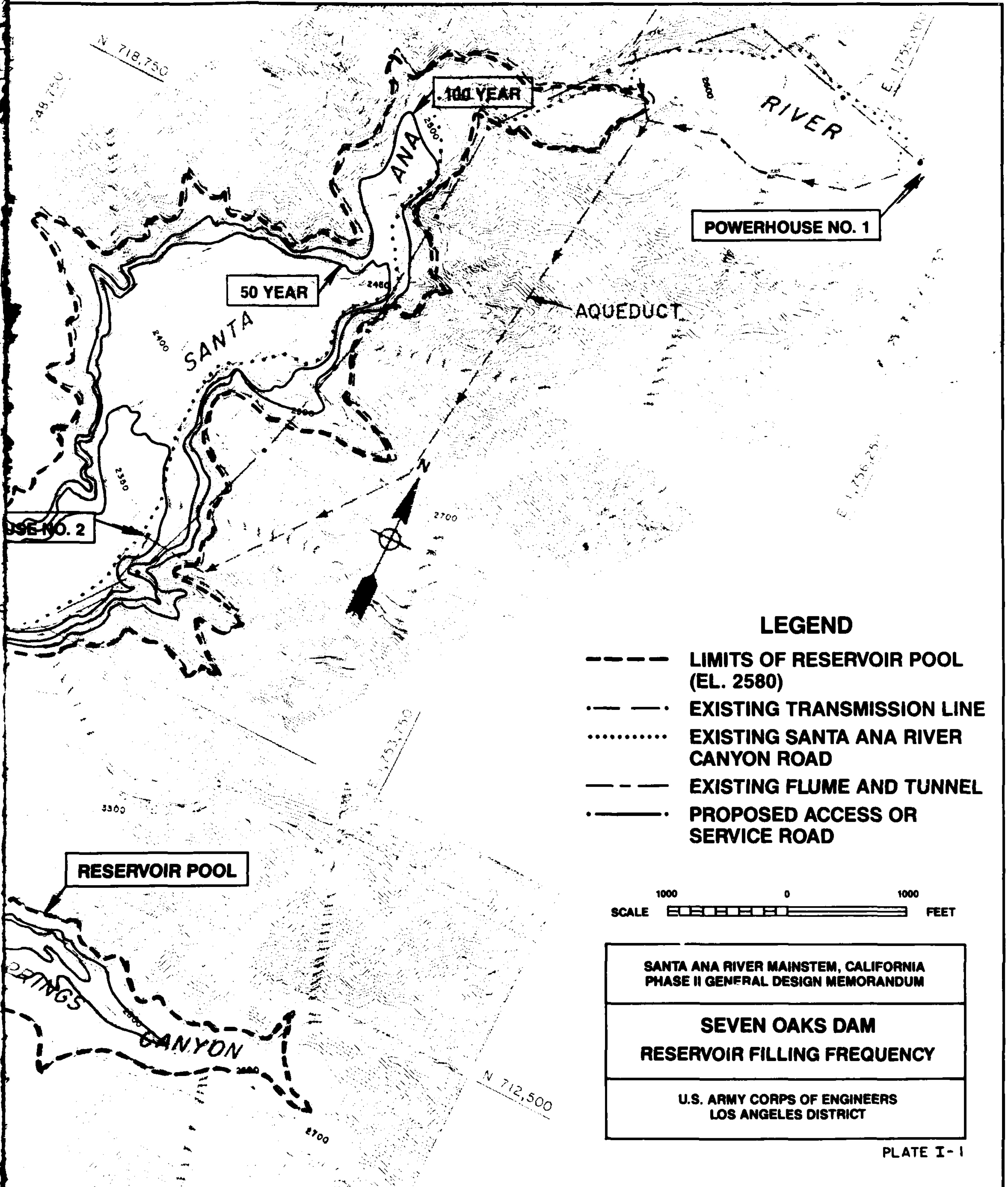
SEPTEMBER 1987 PLATE NO. 6

APPENDIX I


SERVICE ROAD TO INTAKE STRUCTURE
ACCESS ROAD TO TOP OF DAM FROM EXISTING ALDER CREEK ROAD
TOP OF DAM EL. 2610
1V ON 2H
EXISTING ALDER CREEK ROAD
GREENSPOT ROAD
ACCESS ROAD TO OUTLET WORKS
OUTLET WORKS TUNNEL
SANTA ANA
POWERHOUSE NO. 3
1V ON 1H
SPILLWAY CREST EL.
ACCESS RO CONNECT WARM SPR ROAD
DEEP CREEK



UCTURE
ERNMENT
CANYON
SANTA
50 YEAR
POWERHOUSE NO. 2
RIVER
FLOW
10 YEAR
RESERVOIR POOL
WARM
SPRINGS
CANYON
SPILLWAY
CREST EL. 2580
ACCESS ROAD TO
CONNECT WITH EXISTING
WARM SPRINGS CANYON
ROAD
EXISTING
WARM SPRINGS
CANYON ROAD
2700
2600
2500
2400
2300
2250
2200
3200



SANTA ANA RIVER MAINSTEM, CALIFORNIA
PHASE II GENERAL DESIGN MEMORANDUM

SEVEN OAKS DAM
RESERVOIR FILLING FREQUENCY

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE I-1

D RESOURCE USE MASTER PLAN

APPENDIX D

SEVEN OAKS DAM

RESOURCE USE MASTER PLAN

U.S. ARMY CORPS OF ENGINEERS

LOS ANGELES DISTRICT OFFICE

November 1987

CONTENTS

Contents (Continued)

Figures

Attachments

I. INTRODUCTION

Purpose

1-01 The purpose of the Resource Use Master Plan is to establish levels of future land use for the project lands of the Seven Oaks Dam Flood Control project. This report shall identify attainable options for resources use consistent with resource capability and specific project resource use objectives. An important aspect of the plan is to identify and define proper management procedures for the initial construction stages of the flood control project through the ongoing process of resource use project implementation. This will ensure the future availability of the resources for the integration of resource uses consistent with plan recommendations.

1-02 The Resource Use Plan does not address the construction itself of the Seven Oaks Dam or environmental impact mitigation for the project. In formulation of the plan, the Flood Control Project is considered as an existing condition of the project lands with significant environmental mitigation measures already addressed.

Flood Control Project Description

1-03 Seven Oaks Dam is an earth and rockfill flood control dam project to be built on the upper Santa Ana River in the foothills of the San Bernardino Mountains near San Bernardino, California. The project has been designed and engineered by the U.S. Army Corps of Engineers (COE) as part of comprehensive flood control improvements for the entire Santa Ana River Mainstem. The Santa Ana River project is authorized by the Water Resources Development Act of 1986 with Seven Oaks Dam subject to Section 903(B) of the Act.

1-04 The damsite is one of two major flood control storage sites along the river mainstem. Prado Dam Basin in Riverside County is an existing flood control storage site also located on the mainstem and near the intersection of the Orange, Riverside and Los Angeles County boundaries.

Prado Basin is downstream from the Seven Oaks damsite, but is scheduled for extensive flood control improvements as part of the Santa Ana River mainstem program.

1-05 The Seven Oaks Dam and reservoir are located almost entirely within San Bernardino National Forest boundaries. There are private tracts downstream from the damsite. Land purchases or easements will be undertaken as necessary in order to expedite construction and operation of the Federal project. All lands transferred to the Army Corps of Engineers (Department of the Army) through land transfer (i.e., National Forest area), title acquisition, or granting of easement of right-of-way for purposes of access are considered to be "Project Lands." The scope of the Resource Use Plan guidelines includes only those lands designated as "Project Lands."

Executive Summary

1-06 The Resource Use Master Plan identifies appropriate land uses for all locations within the Seven Oaks Dam project area based on the Corps of Engineers' resource use objectives and their relationship to existing resources and land uses. The plan follows the recommendations of the Phase I GDM for the Upper Santa Ana River Flood Storage Alternatives Study and specifically appendix G of that document which has formulated recreation objectives. As the project lands are almost fully within the San Bernardino National Forest, it was agreed through a Memorandum of Understanding that all planning decisions would be consistent to the fullest extent with U.S. Forest Service (USFS) management objectives for the area.

1-07 A recreation market analysis undertaken as part of appendix G concluded that there is currently a high unmet demand for recreation facilities in the vicinity of the project area. The recommended plan, prepared in conjunction with the USFS, proposed development of low density recreation uses at the damsite which would result in a net benefit-to-cost ratio. The USFS had requested the COE to study in lieu site recreation development in addition to on-site recreation considerations. However, further study of in lieu recreation development, as requested by the USFS, was not undertaken because it is not in accordance with current COE policy.

1-08 The San Bernardino National Forest Land and Resource Management Plan (draft), issued by the Forest Service, details management objectives for all parts of the national forest. The specific goals for the Seven Oaks Dam project area were summarized in a statement of April 7, 1987 which calls for only dispersed (low density) recreation uses. It was unclear if the development proposed by the Corps of Engineers was, in fact, in conformance with the USFS objectives until September 18, 1987 when a meeting was held between the two agencies. A conclusion was formed that intensive use areas such as the developed picnic area described in Appendix G would not be within the management

plans of the USFS and therefore would not be considered at this time. The emphasis at this time shall remain on "dispersed" recreation opportunities as described in the USFS Management Plan and on page 5 of this report. This is summarized in a USFS statement to the Corps of Engineers on October 13, 1987 (see appendix).

1-09 Although the intensive use recreation areas are not currently under further study for the damsite, the Resource Use Master Plan has identified five such locations within the flood control project area for the purpose of future consideration if management objectives or operational uses (e.g., maintaining of a permanent water filled reservoir for water conservation purposes) changes at a later date. The selected locations follow the development program recommended in appendix G. The Master Plan also locates areas of low density recreation use, which is consistent with USFS management objectives, along with a Wildlife Management Area, Natural Area, Borrow Area and Project Operations.

Previous Resource Use Planning Actions

1-10 During the selection process for Seven Oaks damsite, recreational use of the project area was identified as project purpose in addition to the flood control protection benefits provided by the project. It has been shown in identifying recreation use as a project purpose that there is an overall recreational net benefit-to-cost ratio of 1.6 to 1 by developing certain recreational facilities in the project lands. Recreational use to a limited degree is also consistent with USFS management objectives for the area as they are listed in the U.S. Forest Service Management section of this report.

1-11 The recreational facilities identified appendix G of the Phase I GDM supplement include a limited capacity day use picnic and parking area with the opportunity for access to the river bed, trail heads and other features in the Upper Santa Ana River canyon.

1-12 The Resource Use Plan prepared in this report does not address specific facility development as in appendix G, but instead defines use zones and land use intensities for use as a framework for future land use decisions.

1-13 The resource use objectives and land uses defined in appendix G play an important role in the preparation of the Resource Use Plan. Formulated through a joint effort by the Corps and the U.S. Forest Service, these previously established objectives provide a basis for the resource use objectives prepared for this Resource Use Master Plan.

Management and Agency Coordination

1-14 The Seven Oaks damsite and the upstream area are located within National Forest boundaries. A Memorandum of Agreement by the Secretaries of the Army and Agriculture was executed on August 13, 1964, pursuant to the management of land and water resources at Corps water development projects located within or partly within the National Forest system. The memorandum states that "Programs of both agencies concerning land procurement, resource development and use, access facilities, roads and trails, on and adjacent to reservoirs and on National Forest lands within the reservoir zones of influence will be correlated to the fullest possible extent...."

1-15 The Forest Service has recently completed the San Bernardino National Forest Land and Resource Management Plan - Draft. Based on the goals of the Memorandum of Agreement, the Seven Oaks Dam Resource Use Plan is to be consistent with and adhere to the standards and guidelines set forth in the U.S. Forest Service Draft Management Plan.

1-16 The Corps of Engineers and the U.S. Forest Service staff have coordinated continuously in the development of the Resource Use Plan. The previously mentioned Memorandum of Agreement identifies the management responsibilities for project lands after project completion. With approval of this plan and in subsequent actions governed by the plan, the USFS will retain full management control over non-flood control operations and maintenance activities within the COE project lands in cooperation with the Corps of Engineers. A Memorandum of Understanding will be executed between the Department of the Army and the Department of Agriculture prior to project commencement to address specific management and coordination concerns.

Area of Study

1-17 Corps of Engineers policy allows for COE project boundaries to incorporate only those areas specific to the Flood Control Project construction, operations, and maintenance. These project lands typically include access roads, utility corridors, the area of the dam structure and maintenance facilities and the high water level elevation of the FCP maximum storage capacity.

1-18 Future projects or resource uses instituted within project lands must be consistent with the Resource Use Plan and adhere to the guidelines set forth. The location of the project lands within a larger environmental setting requires the consideration of factors beyond the project lands in formulating the plan. An analysis of the relationship of the resource elements on the adjacent non-project lands to the project land is undertaken as a part of the project scope. The extent of the area of study in this case specifically includes the watershed area immediately surrounding the project lands. As identified jointly by COE and USFS staff, the specific analysis of this non-project land area is limited to only those key resource elements as having particular importance to the plan formulation. They are discussed further in the resource elements section of this report.

II. U.S. FOREST SERVICE MANAGEMENT OBJECTIVES
OVERVIEW - SEVEN OAKS DAM

2-01 The San Bernardino National Forest and the Resource Management Plan - Draft is a key governing document in the development of resource use designations for the project lands. The location of the project within National Forest boundaries requires that all Corps of Engineers Resource Use Plan objectives and guidelines be consistent with U.S. Forest Service management objectives for the area. The USFS management objectives hope to achieve passive recreational use in the canyon while protecting the wildlife and vegetative resources of the area. Minimum road access is desired for the canyon area although existing uses such as hydroelectric generation would be maintained indefinitely. The USFS management objectives for the project were discussed at the USFS interdisciplinary team meeting on March 24, 1987 and are summarized in the following statement issued April 7, 1987 by the USFS.

Land Use Objectives

RECREATION

- Non-motorized dispersed (recreation only).
- Expand dispersed area opportunities by constructing new riding and hiking trails. Maintain trails at a moderate maintenance level. Construct new feeder access trails, trailheads, and associated facilities. Opportunities include viewing scenery, photography, hiking, horseback riding, bicycling, auto touring, camping, picnicking, gathering forest products, nature study, interpretive services, hunting, fishing, and swimming. Off Highway Vehicle use is specially not allowed.
- Maintain VQO (visual quality objective) (modification, partial retention).

WILDLIFE

- Restrict vehicle traffic to reduce disturbance to wildlife.

- Improve wildlife and fish habitat.

CUSTODIAL

- Protect and improve habitat for sensitive, rare, or threatened and endangered species.

- Protect cultural resources.

Road Use Objectives

RECREATION

- Construct, maintain, and operate Forest Roads and trails to access recreation opportunities.

WILDLIFE

- Reduce the number of roads open to public access in key wildlife habitat areas.

LAND USE

- Maintain/Authorize continuous access for operation and maintenance of hydroelectric facilities.

- Plan for minimum access required for relocation of hydroelectric transmission lines.

- Plan for minimum access to potential sand and gravel material generated from debris pool created by dam.

2-02 The Resource Use Objectives identified by the Corps of Engineers for the Resource Use Master Plan are by COE regulations more generalized in content than the USFS management objectives, but are in full conformance with U.S. Forest Service Management Objectives.

III. RESOURCE USE OBJECTIVES

3-01 The previously stated goal of the Resource Use Plan is to provide for uses of the project land while preserving its wildlife habitat and cultural resource values. The plan identifies attainable options for resource use as determined by analysis of resource capability and consistency with project formulated objectives.

3-02 Resource use objectives are guidelines developed in conjunction with the resource base analysis and resource use suitability plan. Together these provide a basis for meeting the stated resource use plan goal. The resource use objectives provide a comprehensive approach to resource use planning rather than to focus on detailed design or specific uses.

3-03 The resource use objectives identified in this plan have been formulated as a result of Resource Use Objectives identified in Appendix G - Recreation of the Upper Santa Ana River Flood Storage Alternatives Study Supplement to Phase I GDM on the Santa Ana River Main Stem. The resource use objectives are consistent with the USFS Draft Management Plan for San Bernardino National Forest which is a governing document and with ER 1105-2-167 entitled "Resource Use Establishment of Objectives."

3-04 The Resource Use Objectives for Seven Oaks Dam are as follows:

a. Recreation - The project is within easy vehicular range of a large and socio-economically broad population. There is currently a substantial unmet public demand for a wide range of recreational pursuits with the adjacent recreation market area.

   1. To provide a diversity of general recreation opportunities.

   2. To provide affordable recreational opportunities.

3. To provide convenient recreational opportunities for Los Angeles metropolitan recreationists with resultant energy savings by reducing travel to remote and further destinations with similar recreational opportunities.

b. Biological Resources - The project area lies within significant wildlife habitat areas of the upper Santa Ana River and tributaries. The existing vegetation is a key factor in creating habitat value. Endangered plant species also exist within the project area.

1. To provide fish and wildlife resource management including preservation and enhancement with particular attention to endangered species habitat.

2. To provide vegetation management which preserves and enhances wildlife habitat and protects endangered plant species.

3. To provide enhanced vegetation treatments in conjunction with recreational use areas and vegetation and wildlife habitat requirements.

c. Cultural Resources - There exists significant archeological sites as well as historical properties within the project and adjacent areas.

1. To preserve and protect significant cultural resources remaining within the project and adjacent areas.

d. Visual Quality - The project area, after implementation of the flood control project, will still contain significant rural character and high visual quality values.

1. To preserve and enhance the rural pastoral open space and natural esthetic quality and character of the canyon.

IV. RESOURCE BASE ELEMENTS

4-01 The following discussion of physical, biological, and cultural resource elements describes the existing conditions and features of the canyon that will be affected by the project implementation but will also determine other future use opportunities. Variables that will play an important role in the determination of land use, have been mapped to illustrate their interrelationships and to determine a set of opportunities and constraints that will be the base for the suitability study.

Vegetation

4-02 The project area consists of the floodplain, located on the canyon floor, and the upland, found on the slopes and terraces above the floor. Each of these areas supports distinct plant communities, although through the overlap of vegetation species the actual boundaries are often not clearly defined. This interface of the floodplain and the upland occurs typically near the base of the canyon walls. The resulting ecotone is a valuable habitat combining plant groups from both the floodplain and the upland.

4-03 The floodplain consists largely of an open wash area characterized by boulders and cobble and scattered vegetation. The common species in the wash area including California Buckwheat (Eriogonum fasciculatum), Scale Broom (Lepidospartum spuamatum), Mulefat (Baccharis glutinosa), and Sweetbrush (Bebbia juncea). Limited areas of riparian habitat occur in the floodplain in areas protected from normal flooding. A few of these areas form dense stands of Willows (Salix laevigata), California Sycamore (Platanus racemosa), Cottonwood (Populus fremontii) and in limited areas, White Alder (Alnus rhombifolia) and Tamarisk (Tamarix ramosissima). The riparian community is considered the most valuable in the forest as it provides shelter, forage, and breeding areas for both resident wildlife and upland animals. Approximately 70 percent of the wildlife species depend to some extent on the riparian community.

4-04 The upland zones of the project area include the coastal sage community on the drier south facing slopes and chaparral where more favorable soil conditions and less direct sun exposure occurs. The lower growing coastal sage vegetation consists mainly of Sagebrush (Artemesia californica), Buckwheat and Brittlebush (Encelia farinosa). The Chaparral community is characterized by denser, high growing vegetation and, based on soil acidity, includes three subcommunities indicated by Chamise (Adenostema spp.), Ceanothus (Ceanothus spp.), and Oak (Quercus spp.).

4-05 The project area potentially contains at least three sensitive plant species recognized by the U.S. Fish and Wildlife Service or California Native Plant Society. The Santa Ana River Woolly Star (Eriastrum densiflorum sanctorum), and the Slender-Horned Spineflower (Centrostegia leptoceras) have recently been listed as endangered species. The Woolly Star is limited in range to a 7.5 mile stretch of the floodplain downstream from the damsite, the uppermost stand of this species being within the project area just below the damsite. The Spineflower is identified as a potential inhabitat of the Upper Santa Ana River. A third sensitive plant species, Round Leafed Boykenia (Boykenia rountifolia Parry) has not yet been nominated as an endangered plant; however, it is currently on the California Native Plant Society's watchlist of potentially endangered plants. The Boykenia has been sighted in one location within the project area.

4-06 The construction and operation of Seven Oaks Dam will significantly alter the vegetation currently found on the site and also upstream from the dam. Approximately 150 acres will be permanently buried within the dam itself, and up to an additional 150 acres could be temporarily lost during construction operations. Much of the vegetation within the 100-year flood zone of the dam will potentially be buried by sedimentation or will not survive periodic immersion causing deterioration of roots and stems or changes in soil conditions brought on by the sediment. In addition, a 3,000 foot long strip of floodplain directly upstream of the dam may be excavated as a borrow site and would then be devoid of all vegetation.

4-07 The key issues pertaining to vegetation management following the project's construction are the location of the sensitive plant species within the project area and the survival of the remaining riparian community, especially in those areas with existing groves of trees.

Wildlife

4-08 The Upper Santa Ana River canyon floor and slopes contain a variety of habitats supporting a wide range of wildlife species. Many of the species are found in greater numbers relative to similar canyon communities of the San Bernardino National Forest. This is due to most likely from the mingling of the riparian, coastal sage, and chaparral zones to form a rich ecotonal system and also from the relative lack of human intrusion into the canyon. The riparian community, making up only one percent of the national forest, is a principal habitat for many bird and mammal species, providing forage, water and cover.

4-09 The variety of reptiles and amphibians in the canyon is typical of southern California canyons, with many species of lizards and snakes and three types of toads and frogs inhabiting the area. Two significant species, the San Diego Coast Horned Lizard (Phrynosoma coronatum) and Orange Throated Whip Tail (Cremidophorus lyperythus), have been sighted within the project area. These two species are currently given candidate endangered status by the U.S. Fish and Wildlife Service. They both inhabit the floodplain, although the habitat of the Orange Throated Whip Tail can extend onto the side slopes of the canyon.

4-10 Forty-seven species of birds have been sighted during surveys within the canyon, with quantities in upland areas of up to four times that found in other similar parts of the forest. Although the reasons for the number of birds in the project area is unclear, it demonstrates the value of the canyon as a viable bird habitat. The upper portion of the project area also contains one active golden eagle nest according to the Forest Service, and at least two sightings of male Least Bell's Vireos, a bird with proposed status on the Federal endangered species list, have been made in riparian groves in recent years. Although it is felt that the vireos are not permanent inhabitats of the canyon, it is significant that suitable habitat for these sensitive birds occurs in the area.

4-11 The quantity of rodents and other small mammals in Upper Santa Ana Canyon is below that of other similar areas; however, the diversity of species is typical for this type of environment in southern California. The lack of numbers may be due to suboptimal habitat during the study season (1985), although it is not known if this is a continual condition of the canyon. Several significant mammals are evident in the area including badgers, which in small numbers inhabit lateral canyons, beavers, which have been evident in Warm Springs Canyon, and the ringtail, a highly reclusive and nocturnal mammal associated with ridges and cliffs. Badgers are currently under consideration for listing as a species of special concern in California due to diminished habitat. Ringtails which have been spotted in the vicinity of the river are fully protected under state law.

4-12 The mule deer is the most important large mammal found in the project area. The upland areas provide winter range for approximately 135 deer with densities that are among the highest in San Bernardino National Forest. During the summer deer feed in lateral canyons and riparian areas where water can be found. The lateral canyons also are used by deer for fawning areas and migratory corridors, due to the thick cover typically found there. The main canyon and adjoining areas contain a favorable mixture of habitat types that support both a migratory and resident mule deer population.

4-13 The Upper Santa Ana River is normally perennial except at the confluence of Warm Springs Canyon, where it can become dry during drought years. A self-sustaining population of brown trout occurs in the stream, as there has been no stocking of these fish since the 1960's. Rainbow trout, a fish stocked in the higher regions of the river also are present in the canyon. Relative to its normal size the river appears to be very active in terms of trout production.

4-14 As the construction of the Seven Oaks Dam removes several hundred acres of vegetation, and the subsequent flooding and sedimentation expands this process, a loss of valuable wildlife habitat will, in turn, occur. Much of this lost habitat will be within the floodplain and the highly valuable ecotonal and riparian zones. This includes habitat for the two sensitive reptile species and forage and breeding areas for many mammals and birds. In addition to the habitat losses from the dam itself any increase in public access following the project could result in wildlife harassment from dogs, off-road vehicles, poachers, and target shooters.

4-15 Due to the dam's impact on the canyon's environment, the importance of remaining lateral canyons as migratory routes, fawning areas, and feeding areas for mule deer will increase. The floodplain that remains above the sedimentation zone will also increase in value as a habitat for the sensitive reptiles and other wildlife of that zone.

Topography

4-16 The topography of the Upper Santa Ana River canyon is characterized by the rolling alluvium of the wash area formed by the river's channel and the steep slopes rising hundreds of feet above the canyon floor. Although the occasional alluvial fans formed at the base of the slope and a few knolls within the upland area provide transitional topographic zones, the majority of the terrain above the canyon floor is at least a 50 percent slope.

Geology

4-17 The significant geologic features within the project area include the eroded and transported clastic material that forms the river bed, the alluvial deposits as noted under Topography, and some limited areas of potato sandstone formation in Warm Springs Canyon. The clastic material, ranging in size from the fine sands to truck sized boulders forms a layer of 50 to 90 feet thick over the existing bedrock in the canyon.

4-18 The construction of the dam could potentially provide investigative opportunities for geologists due to the masssive cutting of existing slopes involved; however, the potato sandstone, a formation with a high potential for containing vertebrate fossils and is of interest to geologists occurs far above the construction site. If a spillway access road is cut parallel to Warm Springs canyon, cross sections of the potato sandstone could be exposed and be available to geological investigation.

Water Quality

4-19 The current quality of water in the canyon is generally excellent for this area with the exception of high coliform levels found near the headwaters of the stream several miles above the project area. Some deterioration in quality could be expected with increased human activity in the area but could be mitigated with careful management of human waste and trash. A temporary condition of higher mineral content and possibly contamination from equipment and vehicles may occur during the construction of the dam, but the former quality should return upon the dam's completion.

Cultural Resources

4-20 The Corps of Engineers is required to investigate and identify the presence of cultural resources within the project area and is authorized to take mitigation measures for those resources that would be affected by construction. These resources, which include structures, sites, artifacts, and other information provide evidence of historic and prehistoric human habitation or use of the area.

4-21 Surveys of the project area have identified 38 sites to be considered as cultural resources. Several of these are facilities still in use today, including the Southern California Edison Powerhouse and aqueduct system, the Francis Cuttle Weir Dam, the Orange Avenue Bridge, the North Fork Canal and weir system and the Southern California Edison transmission lines. Other resource sites include remnants of an early road through the canyon, existing foundations of past structures built in the canyon, miscellaneous artifact sites, bedrock metate sites, and a seeping hot springs. In addition, an early grave site of two stillborn children exists within the damsite and will require relocation prior to the construction. The cultural resource sites are scattered throughout the project area and are subject to different magnitudes of impact from the dam, which correspond to the flood inundation levels.

4-22 Of the 38 sites in the project area, 19 have been assessed by the Corps of Engineers as having little or no significance. The remaining sites can be divided into the following categories for resource management based on the potential significance of each site and the extent of potential impacts from the dam:

a. Sites potentially important for preservation and public historical interpretation program.

b. Site potentially appropriate for preservation but not public access.

c. Sites to be relocated (grave site).

V. EXISTING LAND USE ELEMENTS

5-01 The following land use elements, when compared to the key resource variables noted in the preceding section, will provide a composite of relationships for the investigation of land use suitability.

Recreational Use Patterns

5-02 The most direct access into the canyon is through a gate directly off Greenspot Road. Only authorized vehicles or pedestrians are permitted into the canyon. Although the site is technically open to the public year round, the gated roads at the canyon mouth and in Monroe Canyon, and difficult access from Warm Springs Canyon, restrict most public access to foot traffic only. This restriction and a general lack of public awareness of the site have produced low to moderate recreational use rates in the project area. Although it is not a popular location, the site offers opportunities for hunting, fishing, hiking, picnicking, backpacking, swimming and equestrian uses.

Electrical Generation Facilities

5-03 Southern California Edison manages three hydropower generating stations in the canyon, two of which are within the project area with one of those in the reservoir site itself. In addition, the Edison aqueduct system runs through the canyon feeding water to the three power houses. This system, originally built between 1892 and 1905 consists of flumes, tunnels, sand boxes, weirs, and feeder reservoirs. The generating capacity of the system is to remain unchanged and in operation after the construction of the dam, although portions of the aqueduct system and the affected powerhouse will require modifications. The Edison power lines currently running through the canyon will be relocated at higher elevations to remain outside the inundation levels.

Flood Control Operations

5-04 The proposed Upper Santa Ana River flood control operations will consist of a 550-foot high earth-rockfill dam with a gross storage capacity of 145,600 acre-feet, an unlined trapezoidal spillway, and an outlet works to control flows. The dam is capable of regulating flow during the standard project flood and to retain sediment deposits behind the dam for a project life of 100 years. During the life of the project the system should require only normal maintenance of the gates and equipment and periodic inspections; no dredging is anticipated during this period. Management of the operations will be handled by personnel from a control room. A parking area and access road will be constructed for the control room.

VI. RESOURCE ANALYSIS COMPOSITE

6-01 The following Resource Analysis Composite overlays the key resource elements previously identified and existing land use elements to test the environmental sensitivity of the various interrelationships. The plan, combined with the identification of the resource use objectives will serve to determine a set of criteria for locating suitable areas for specific land uses.

Flood Inundation Levels

6-02 These levels are shown based on the ultimate estimated sedimentation zones (at 100-year project life) at key flood levels. The zones of inundation and subsequent sedimentation are the primary factor in determining an area's suitability for potential use.

a. Debris Pool: (2300') This will be the normal water level following flood operations and will support little use.

b. Ten Year Flood Zone: (2402') This area will come under inundation on a fairly regular basis and is not appropriate for most uses.

c. Fifty Year Flood Zone: (2503') This zone will show impacts of occasional flooding but may also support vegetative habitats during extended non-flood periods. Uses capable of withstanding occasional inundation and uses with shorter than project life spans could occur here.

d. One Hundred Year Flood Zone: (2535') This area may incur very occasional inundation but is appropriate for uses capable of withstanding such infrequent flooding.

e. Project Take Line: (2583') This line has been established as the limit of the project study area by adding 3 feet of freeboard to the spillway crest elevation 2580'. All potential use areas shall occur within this line as part of this project.

Topography

6-03 Slopes less than 3:1 horizontal to vertical (33 percent) are generally considered appropriate for limited development. Areas with gradients less than this are very limited outside the floodplain itself and are valuable for use opportunities.

Vegetation

RIPARIAN ZONE

6-04 The riparian areas outside of the debris basin can potentially provide viable habitat until they are reached by sediment levels. Thus, they become increasingly valuable as management areas as their elevation increases.

RIPARIAN TREE STANDS

6-05 These are important elements for preservation due to their extreme habitat value and their very limited range following the project implementation.

EXISTING ZONE OF SENSITIVE PLANT SPECIES

6-06 These very limited areas must be protected from uses with the potential for creating adverse impacts on the plants.

Cultural Resources

6-07 These sites should be protected from other human uses due to their historical significance. However, specific sites have been deemed appropriate for public interpretation and could provide use opportunities. Resource sites shown in current use can continue to be managed as appropriate (Note: the specific site locations are not for public use and will be excluded from published documents).

Wildlife

ENDANGERED SPECIES HABITAT

6-08 The preservation of these areas is important although low intensity uses within the habitats should not be in conflict due to the reclusive nature of the specific animals. (Note: The specific species locations are not for public use and will be excluded from published documents).

LATERAL CANYONS

6-09 These canyons are very important for preservation as deer habitats and should be avoided as possible by other uses. Canyons should not be bisected by roads, trails or other human uses beyond that which is necessary for the construction and operation of the dam. Although the majority of these canyons extend outside the project area, the confluence of these canyons with the Santa Ana River is of prime significance.

EXISTING GOLDEN EAGLE NESTING SITE

6-10 This very sensitive site should be provided with a half mile radius buffer from human impact. (Note: the specific site location is not for public use and will be excluded from public documents).

FISHERIES

6-11 Potential fishing upstream of the dam will be limited and possibly deleted within the project area depending on future stream conditions. A put and take fishery within the debris basin which would require stocking of the stream may be unfeasible due to variable water conditions.

Water Quality

6-12 Intense use areas should be located away from perennial stream beds to protect water quality.

Construction (Operations and Maintenance)

6-13 The following elements preclude any other preservation or use areas and should be restricted from other non-operational uses for safety and functional reasons:

a. Dam:

b. Spillway:

c. Outlet Works Tunnel, Plunge Pool:

Other construction elements include:

d. Government canyon stream diversion: The stream diversion area occurs within the debris basin; no other uses are anticipated at this site.

e. Haul Road Options: These may produce visual degradation depending on their relationship to other uses and the quality of a post construction mitigation program. The roads may also increase the erosion potential of the canyon walls and the turbidity of the stream water.

f. Access Road Relocation: The project operations access road may present opportunities for other adjacent use areas due to extensive grading that may be required for its construction. However, it may impact sensitive habitat areas as it crosses lateral canyons.

g. Transmission Line Relocation: The relocation of power lines presents a potential for visual degradation from the grading involved; however, the construction process may also create new use opportunities due to new access routes and graded areas. Other uses directly below the lines should be avoided for safety and functional reasons.

h. Deep Creek Spillway Easement: This area should be restricted from other uses due to the overflow potential and its location within a sensitive habitat area.

i. Excavation Area: This potential borrow site will be fully within the debris basin and will not support other uses.

Visual Analysis

6-14 The canyon slopes provide a positive visual image as a pastoral environment left without the impacts of man. These slopes are highly visible throughout the project area and may be impacted from grading operations as noted above. Thus, uses involving stripping and grading should be minimized on the slopes to prevent visual degradation.

VII. RESOURCE PLAN

7-01 The Composite Resource Analysis Plan identifies opportunities and conflicts in the relationships between the various resource elements and existing land uses. A comparison of the composite analysis plan to the resource use objectives for the project begins to define the land use categories appropriate for the project lands. The land use designations and resultant use intensity levels create a framework for future land use decisions. Conflicts will remain within and between the use areas as designated. These conflicts must often be resolved on a case by case basis.

Project Operation Areas

7-02 This includes all facilities, areas and access roads involved with the operation and maintenance of the FCP as well as facilities associated with the operations and maintenance of existing power generating facilities. It also includes historically significant sites and systems that are in current active use such as the power houses, flume system, Orange Avenue Bridge, Cuttle Dam and the canal system.

7-03 This designation identifies severely restricted use areas which essentially exclude other resource use potentials due to the nature of the existing conditions of use and the lack of significant remaining natural resources.

Borrow Area

7-04 This area upstream of the dam structure may be completely modified by excavation of material for construction of the dam embankment. This area is not a part of the restricted FCP operations and maintenance areas but has limited resource value due to the potential loss of all vegetative resources, visual degradation, a constantly fluctuating water

level and continuous sediment build-up. The difficulty of access, safety and water quality concerns and unpredictable water levels reduce the feasibility of the borrow area as a put and take fishery.

Natural Area

7-05 This area designates highly sensitive or endangered plant species areas, wildlife habitats, geologic formations and cultural (i.e., archaeological, historical) areas for preservation or limited use on a very restricted basis. The nature of the areas may allow increased use, including interpretive use of the sensitive resource if certain criteria are met (i.e., after requirements of Section 106 of the National Historic Preservation Act have been fulfilled), or if the conditions change concerning the area's sensitivity.

Wildlife Management Area

7-06 The Wildlife Management Area consists of important and sensitive habitat areas or vegetative resources that require custodial enhancement if possible, or protection from further degradation. Low intensity but highly controlled and appropriately designed uses may take place such as hiking, or other similar passive uses. The vegetative resource areas, in addition to providing wildlife habitat, are important for soils stabilization on slope areas and other highly erodible soil areas. The protection of the vegetative resource areas reduces potential sedimentation in the FCP.

Recreation - Low Density Use

7-07 Recreation-Low Density Use is defined in ER 1120-2-400 as "lands acquired for project operations and allocated for low density recreation activities by the visiting public as required as open space between intensive recreational developments or between an intensive recreational development and land which, by virtue of use, is incompatible with the recreational development and would detract from the quality of the public use. Such incompatible land may be located either on the project or adjacent to the project. Land required for ecological workshops and forums, hiking trails, primitive camping or similar low density recreational use available for a significant role in shaping public understanding of the environment will be under this allocation. No agricultural uses are permitted on this land except on an interim basis for terrain adaptable for maintenance of open space and/or scenic values."

7-08 Since public pedestrian access is allowed into project lands through Warm Springs Canyon, low density recreation use is an acceptable land use opportunity. In ER 1120-2-400, the Wildlife Management Area is defined as being "continuously available for low density recreation activities." Since the Wildlife Management Area can accommodate low density recreation use, there has been no land allocated for the low density use outside of the Wildlife Management Area.

Recreation - Intensive Use

7-09 Prior to the adoption of the USFS management objectives as formulated in the Draft Resource Management Plan (non-motorized dispersed recreation only), the Corps of Engineers had proposed one intensive use site upstream from the damsite. This area as described in appendix G of the Phase I GDM would consist of a 6 acre parking and picnicking site with access to trail heads. In conformance with the current recreation objectives of USFS no intensive use areas will be planned as part of the flood control project. However, as a part of this study, potential areas of intensive use have been located for future reference.

7-10 Intensive Recreation is defined in ER 1120-2-2400 as "developed public use areas," including areas for concession and quasi-public development. If management objectives should change in the future or if the nature of the dam operation should change (i.e., development of a permanent water level for the reservoir), then intensive recreation uses can be considered. This would be consistent with resource use objectives developed by the Corps of Engineers for the project lands. The occurrence of this use area within the project lands is very limited. Flood inundation and sedimentation, environmental sensitivities, prevailing slope conditions and operation and maintenance consideration, among other factors, combine to reduce the potential for extensive public access and use of project lands. Areas that do provide an opportunity for increased use are generally outside of the 50-year storm inundation zone, on suitably level land (33 percent or less slope) are not within range of prohibitive environmental constraints and may be able to take advantage of trail head opportunities or cultural resource interpretive sites in addition to the general natural recreational amenities offered by the Upper Santa Ana River Canyon. An analysis of the specific intensive use sites suggested within the project lands is included as follows:

AREA NO. 1

7-11 This area near the top of the dam provides particular opportunities for viewing of upstream and downstream areas around the dam as well as interpretation of the dam construction and operation itself. The proximity of the dam structure and public access road provide already-developed facilities, thereby limiting additional environmental impact to surrounding areas. The site also presents opportunities for education on the entire Santa Ana River Trail system, which essentially begins at the damsite.

AREA NO. 2

7-12 This area lies above 50-year flood inundation levels and is on a large area of reasonably sloping land. It is also buffered from the sensitive deer migration routes and away from other environmentally sensitive areas. However, road access may be quite difficult and costly due to extensive grading and the site lies above the possible borrow

area, a potential major visual blight. This limits desirability for many recreational pursuits. There are opportunities for trailhead parking, FCP interpretation and possible cultural site interpretation.

AREA NO. 3

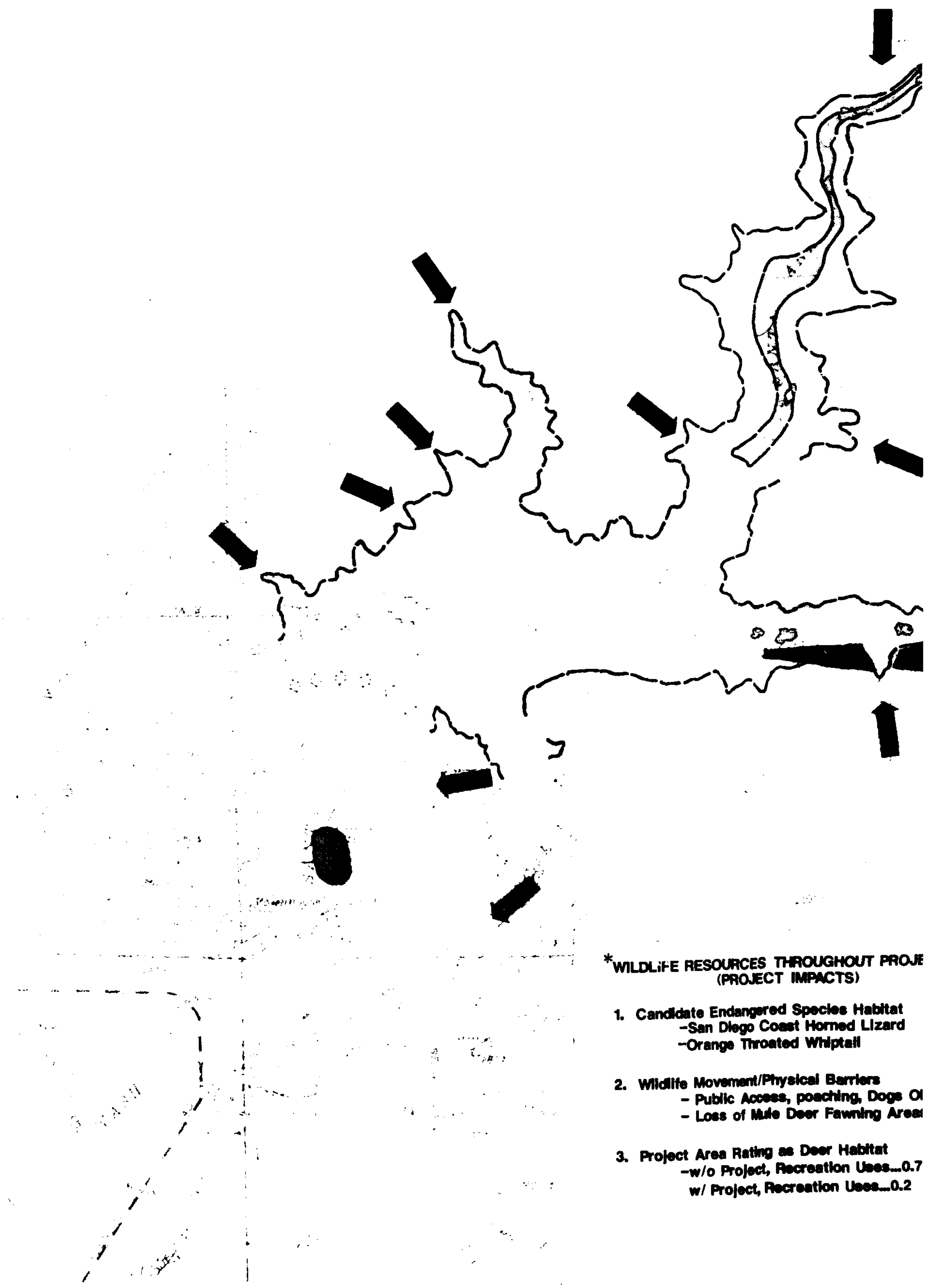
7-13 This site was previously identified in Appendix G - Recreation as a day-use area. It combines many positive site characteristics in terms of use including location above most flood levels and out of sedimentation zones. This site is visually uninteresting and barren with a south-facing aspect and requires an access road to be built across the riverbed from the existing canyon road. As previously recognized, it still displays reasonable potential for limited recreational facilities development in the canyon. This might include a day-use picnic area with portable sanitary facilities, picnic tables and a small parking area as proposed in appendix G.

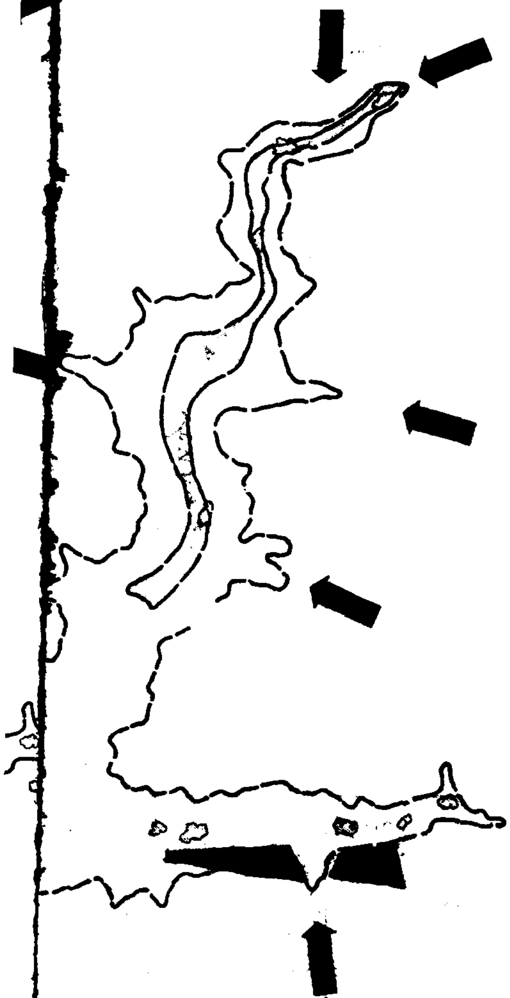
AREA NO. 4

7-14 This area lies near Powerhouse No. 2 and the adjacent Edison employee settlement site. It is anticipated that this area would be appropriate for only very limited interim use as recent studies project that by project year 100 as much as 50 feet of sediment may cover this site.

AREA NO. 5

7-15 This area is located in Warm Springs Canyon which is a significant deer habitat area. The adjacent access road for the operations and maintenance of the FCP and power houses allows for this site to become quite feasible as a use area. It meets resource element criteria for suitability in a manner similar to Area No. 3.


*WILDLIFE RESOURCES THROUGHOUT PROJE
(PROJECT IMPACTS)
1. Candidate Endangered Species Habitat
-San Diego Coast Horned Lizard
-Orange Throated Whiptail
2. Wildlife Movement/Physical Barriers
- Public Access, poaching, Dogs Ol
- Loss of Mule Deer Fawning Area
3. Project Area Rating as Deer Habitat
-w/o Project, Recreation Uses...0.7
w/ Project, Recreation Uses...0.2



LEGEND

GEOLOGICAL RESOURCES

Potato Sandstone Formation
High Potential for Vertibrate Fossils

WILDLIFE RESOURCES*

Lateral Canyons: Critical Locations as Mule Deer Migratory Routes and Fawning Areas

VEGETATIVE RESOURCES

Limit of Proposed Endangered Plant Species

Riparian Zone Outside of Debris Basin

Riparian Tree Stands

Project Take Line 2583'

RE

E RESOURCES THROUGHOUT PROJECT AREA
(PROJECT IMPACTS)

date Endangered Species Habitat
-San Diego Coast Horned Lizard
-Orange Throated Whiptail

Wil

ood

fe Movement/Physical Barriers
- Public Access, poaching, Dogs ORV's, Wildlife Harassment
- Loss of Mule Deer Fawning Areas in Flood Zone

ct Area Rating as Deer Habitat
-w/o Project, Recreation Uses...0.7
w/ Project, Recreation Uses...0.2

DATE

REMARKS

0 1000 3000 5000
Scale in feet

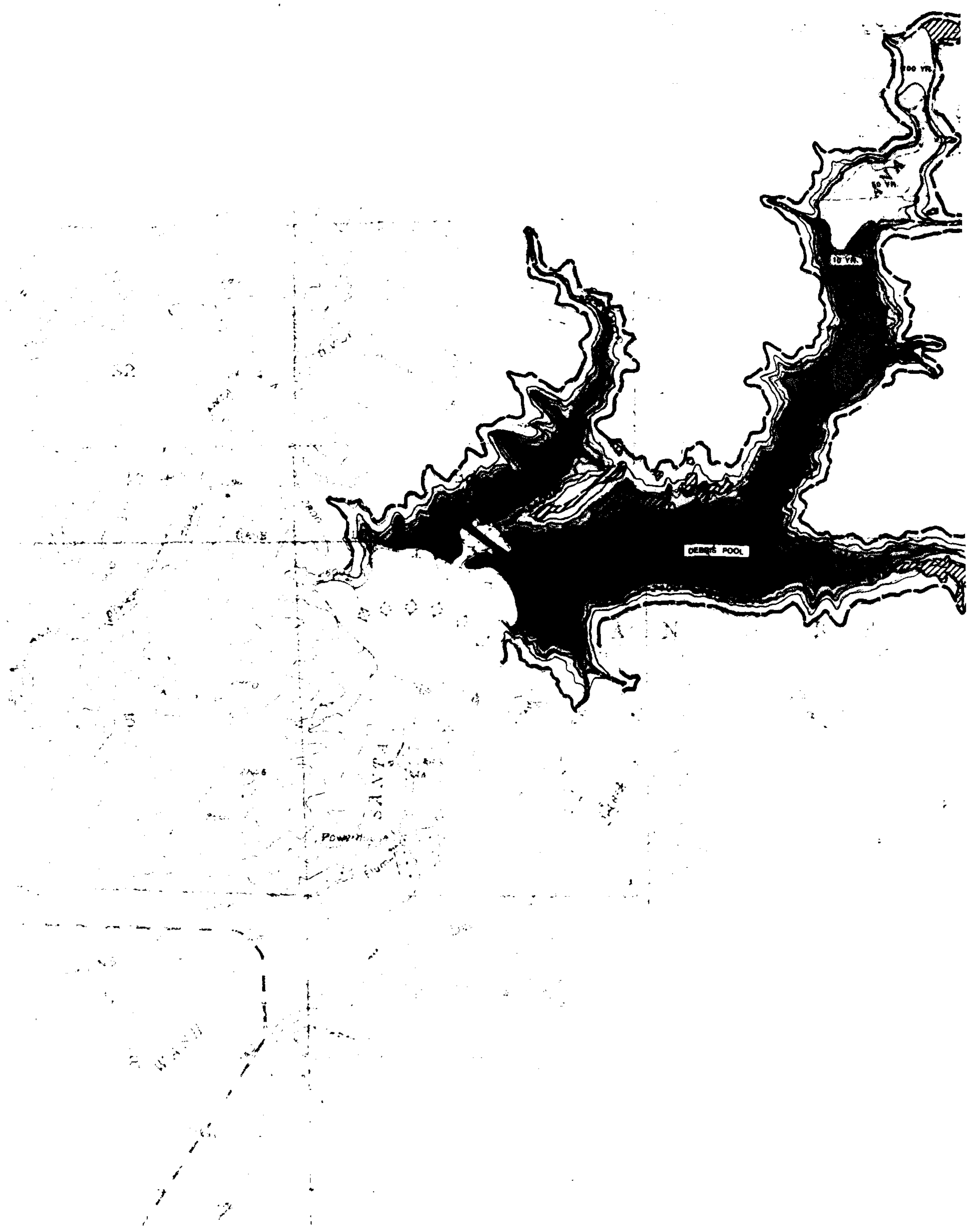
SEVEN OAKS DAM
San Bernardino, California

RESOURCE USE MASTER PLAN

WILDLIFE-VEGETATIVE-AND GEOLOGICAL RESOURCES

US Army Corps of Engineers
Los Angeles District

FIGURE : 1



100 YR.
50 YR.
10 YR.
DEBRIS POOL

LEGEND

10Yr.
50Yr.
100Yr.

Debris Pool
2300'

10 Year Flood Event
2402'

50 Year Flood Event
2503'

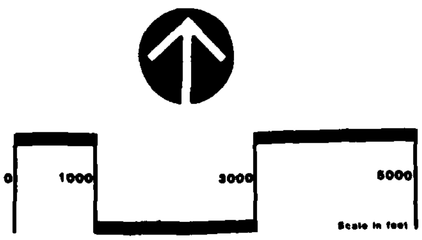
100 Year Flood Event
2535'

3:1 Slopes or Less

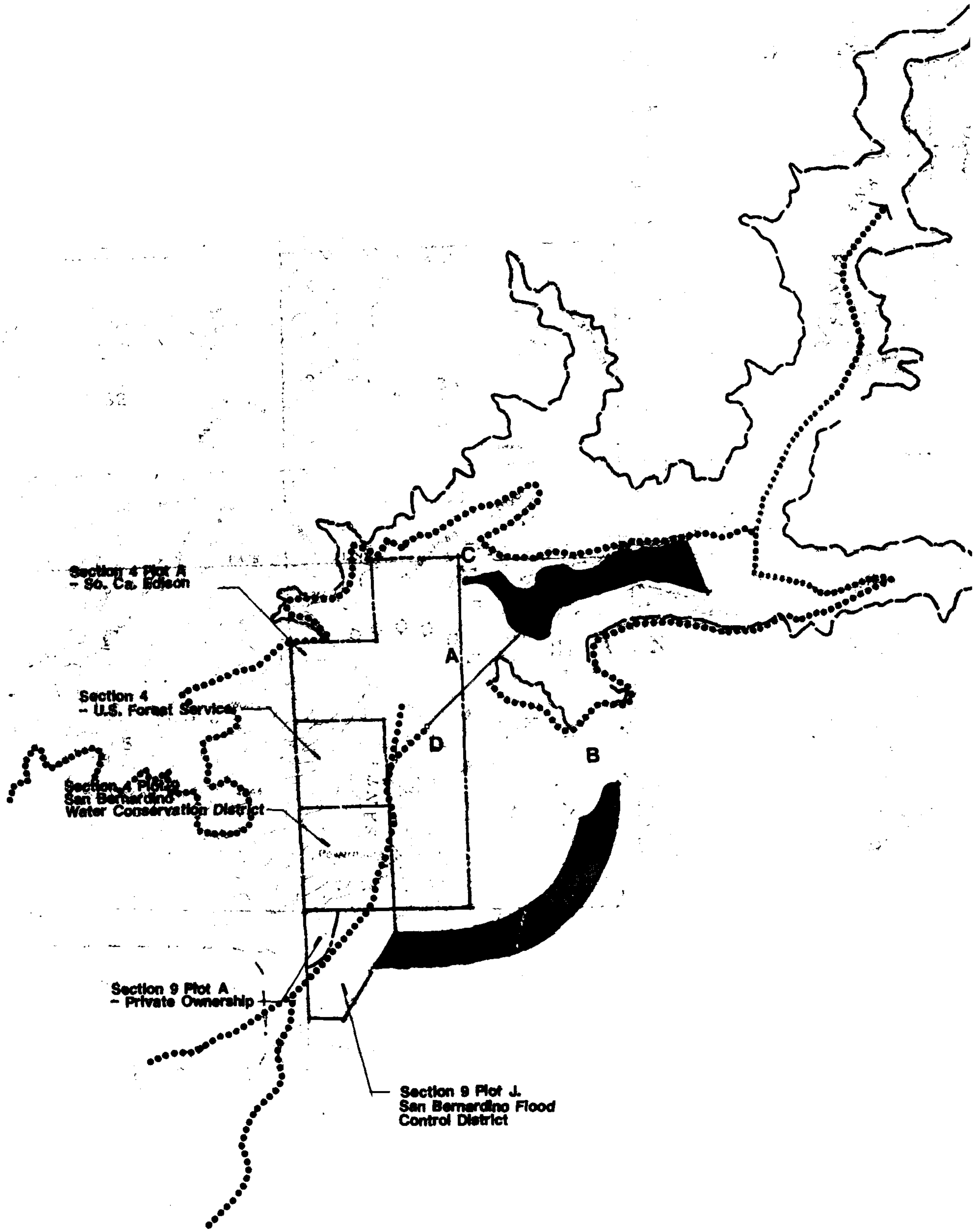
Project Take Line 2583'
Maximum Controlled Reservoir Elevation 2580'
Spillway Crest Elevation 2580'

Flood events are for future conditions with 100 year sediment in the reservoir.

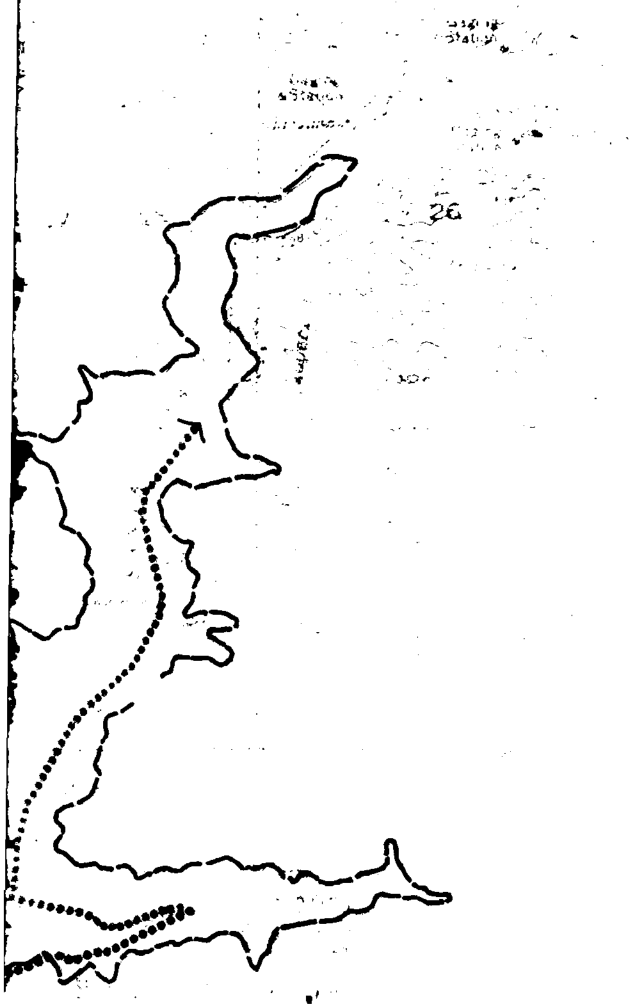
DATE	REMARKS



SEVEN OAKS DAM San Bernardino, California	
RESOURCE USE MASTER PLAN **FLOOD INUNDATION LEVELS TOPOGRAPHY**	
US Army Corps of Engineers Los Angeles District	FIGURE: 2



Section 4 Plot A
- So. Ca. Edison
C
A
Section 4
- U.S. Forest Service
D
B
Section 4 Plot
San Bernardino
Water Conservation District
Section 9 Plot A
- Private Ownership
Section 9 Plot J.
San Bernardino Flood
Control District



LEGEND

CONSTRUCTION

(black square)	Borrow Site
A	Dam
B	Spillway
C	Government Canyon Ridge Notch – *Reroute of Stream*
D	Outlet Works Tunnel / Plunge Pool
(shaded square)	Deep Creek Overflow Easement
(dotted line)	Proposed Construction Access
(dotted line)	Existing Construction Access
(dashed line)	Project Take Line 2583'
(dash-dot line)	Property Line

DATE

REMARKS

0 1000 3000 5000

Scale in feet

SEVEN OAKS DAM

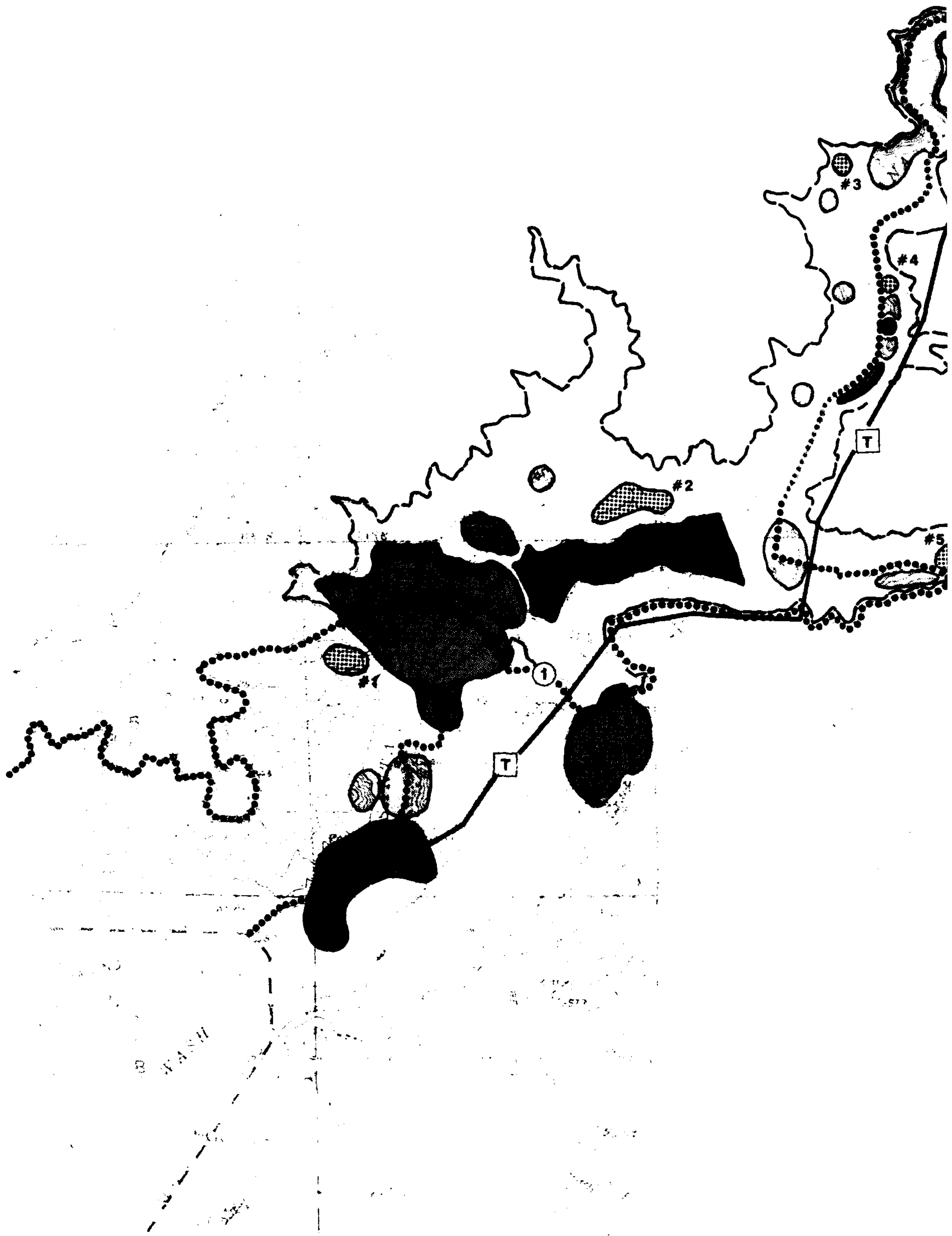
San Bernardino, California

RESOURCE USE MASTER PLAN

CONSTRUCTION LAND OWNERSHIP

US Army Corps of Engineers
Los Angeles District

FIGURE: 3



#1
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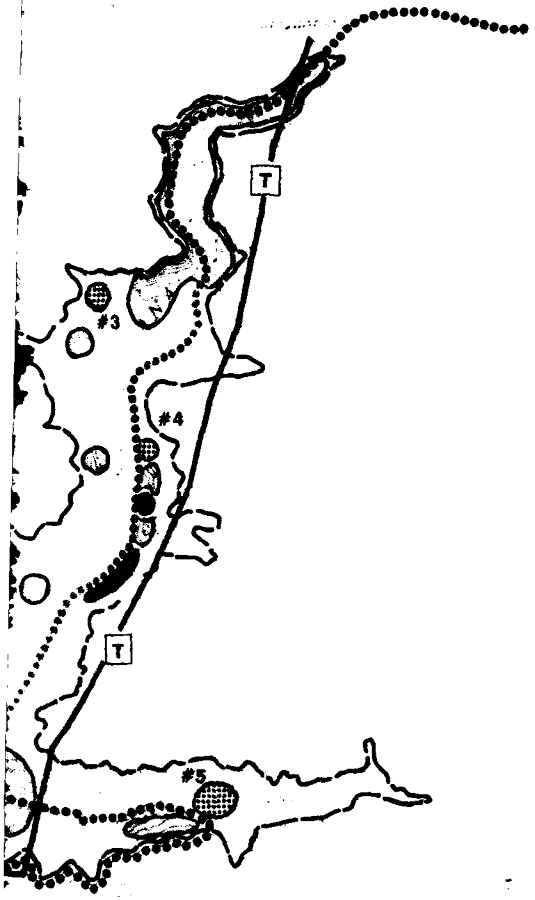


LEGEND

Borrow site

Project Operations

Wildlife Management Area
(Includes land use allocation for Recreation - Low Density Use)

Natural Area

Recreation - Intensive Use Area
(For future reference only - not under current consideration for land use allocation. For current allocation in these areas see Wildlife Management Area.)

Transmission Line Location

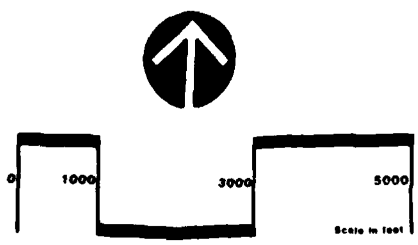
Proposed Operations Access

Existing Operations Access

Project Take Line 2583

NOTE: NUMBERS REFER TO DESCRIPTION IN TEXT

DATE REMARKS



SEVEN OAKS DAM
San Bernardino, California

RESOURCE USE MASTER PLAN

LAND USE SUITABILITY PLAN

US Army Corps of Engineers
Los Angeles District

FIGURE: 4

ATTACHMENT

PUBLIC LAWS

The following laws provide guidance for the development and management of Federal projects for various purposes according to the intent of Congress and as they apply to the Seven Oaks Dam and Vicinity Flood Control Project.

a. Water Resources Development Act of 1986, provides authorization and funding for a nation-wide agenda of water resource development projects, including the Upper Santa Ana River Mainstem Flood Control Project.

b. Section 4, Public Law 78-534 (The Flood Control Act of 1944), as amended, authorizes the Corps of Engineers to construct, maintain, and operate public park and recreational facilities at water resource development projects and to permit local interests to construct, maintain, and operate such facilities.

c. Public Law 85-624 (The Fish and Wildlife Coordination Act of 1958) provides for the integration of fish and wildlife conservation in water resource projects. The U.S. Fish and Wildlife Service has prepared a report, dated 1 February 1978, pertaining the fish and wildlife resources of the project area. The recommendations and findings contained in the report have been given full consideration.

d. Public Law 89-72 (The Federal Water Project Recreation Act of 1965), accompanied by House Committee Report 154, requires that the project consider inclusion of outdoor recreation and fish and wildlife enhancement. It also provides for non-Federal participation in land acquisition and in the development and management of project recreational facilities and fish and wildlife resources.

e. Public Law 91-190 (The National Environmental Policy Act of 1969) requires that the environmental effects of each project and the means and measures to minimize any adverse effects to be evaluated and presented in an environmental impact statement (EIS).

f. Public Law 96-515 (The National Preservation Act of 1966, as amended) requires that prior to the approval of the expenditure of any funds on an undertaking or the issuance of a license or permit, the Federal agency shall take into account the effect of the undertaking on any property included in or eligible for inclusion in the National Register of Historic Plans and shall afford the Advisory Council on Historic Preservation a reasonable opportunity to comment.

g. Public Law 93-291 (The Archeological Recovery Act of 1960, as amended) provides for the preservation of historical and archeological data which might otherwise be irreparably lost or destroyed as the result of any alteration of the terrain by any Federal construction project or federally licensed project, activity or program.

h. Public Law 91-604 (Clean Air Act of 1977, as amended) requires any activity which may result in discharge of air pollutants must comply with Federal, state, interstate, and local requirements respecting control and abatement of air pollution.

i. Public Law 93-205 (Endangered Species Act of 1973) amended 1978, 1979, 1982, requires Federal agencies, in consultation with the secretary of the Interior/Commerce, to utilize their authorities in furtherance of the purposes of the Act by carrying out programs for conservation of endangered and threatened species protected by the Act. Section 7(c) specifically requires Federal agencies to request information from U.S. Fish and Wildlife Service and National Marine Fisheries Service on endangered and threatened species that may be in the proposed project area. If listed species may be present, Federal agencies are required to prepare an environmental assessment to identify any listed species likely to be affected by the project.

j. Public Law 95-217 (Clean Water Act of 1977, as amended) requires that sites for the discharge of dredged or fill material be specified through the application of EPA guidelines. Furthermore, discharge of dredged or fill material into waters of the United States would require application for the Section 404 permit or preparation of a 404(b)(1) water quality evaluation.

RELATED PUBLIC DOCUMENTS

The following documents have played an integral role in the preparation of this plan report:

a. Phase I GDM - Santa Ana River Main Stem including Santiago Creek - Army Corps of Engineers.

b. Flood Control Alternatives Study - Supplement to Phase I GDM (includes appendix G - Recreation) - Army Corps of Engineers.

c. San Bernardino National Forest Land and Resource Management Plan - Draft USFS Pacific Southwest Region, U.S. Department of Agriculture.

d. Preliminary Prado Dam Basin Land Use Analysis Report - Santa Ana River Main Stem including Santiago Creek Phase I GDM - Army Corps of Engineers.

e. Resource Use Plan for New River Dam, Design Memorandum No. 3 Phase II GDM, Gila River Basin, 1982, Army Corps of Engineers.

APPLICABLE REGULATIONS

a. ER 1120-2-400 Recreation Resources Planning prescribes policies, guidelines, procedures and definitions for insuring that protection and enhancement of recreation resources are given equal treatment with other objectives in the planning and development of water resource projects under the jurisidiction of the Corps of Engineers. A key feature of this regulation is the definitions provided for land use allocation categories applicable to Corps projects including low density and high intensity recreation uses (see Section 7.03 a. and b. for definitions).

b. ER 1130-2-400 Project Operation - Management of Natural Resources and Outdoor Recreation at Civil Works Water Resource Projects (June 1986). This regulation provides policy and procedural guidance for the administration and management of civil works water resource projects only. General policies regarding planning, authorization, development and construction civil works projects are contained in references and in other regulation and policy statements.

c. ER 1165-2-400 Recreation Planning, Development and Management Policies (August, 1985). This regulation defines the objectives, philosophies and basic policies for the planning, development and management of outdoor recreation and enhancement of fish and wildlife resources at Corps of Engineers water resource development projects.

d. ER 1105-2-50 Environmental Resource Planning (August 1984), provides requirements for addressing environmentally sensitive planning techniques consistent with the National Environmental Policy Act of 1969 and project lands.

e. ER 1105-2-167 Establishment of Resource Use Objectives (April 1978), provides policy and guidance for establishing resource use objectives for all civil works water resource projects.

f. ER 200-2-2 Environmental Quality: Policy and Procedures for Implementing NEPA (August 1980). This regulation provides policy and procedural guidance to supplement the Council on Environmental Quality (CEQ) final regulations implementing the procedural provisions of NEPA.

USFS STATEMENT, OCTOBER 13, 1987

The following letter addressing the USFS management policies for the Seven Oaks Dam was issued to the Corps of Engineers subsequent to an inter-agency policy meeting on September 18, 1987.

DEPARTMENT OF AGRICULTURE | FOREST SERVICE | San Gorgonio Ranger District | 34701 Mill Creek Rd. Mentone, CA 92359

ERB

Reply To: 2540

Date: October 13, 1987

Mr. Robert S. Joe
Chief, Planning Division
U.S. Army Corps of Engineers
Los Angeles District
300 N. Los Angeles Street
P.O. Box 2711
Los Angeles, CA 90053

Dear Sir:

In pursuit of the planning for the Seven Oaks Dam element of the Upper Santa Ana River Flood Storage Project, The Corps of Engineers is preparing a Resource Use Master Plan. The focus of this particular plan is on the management of project lands once the dam is completed. In regard to National Forest within the project area, the Resource Use Master Plan should reflect the direction contained in the draft of the San Bernardino National Forest Land and Resource Management Plan. Mr. John Williams of your office has previously been provided with a copy of this document.

In regard to recreation management of the National Forest within the project area, the draft Plan prescribes an emphasis on "dispersed" recreation opportunities. This is compatible with the currently proposed design of an earth and rock filled dam with seasonally operated debris basin. The Forest Land and Resource Management Plan will be subject to periodic review and revision. Also, any changes in the design or operation and maintenance of the Seven Oaks Dam may require reassessment of National Forest management objectives within the project area.

If you have any questions regarding the management objectives of National Forest within the project area or any other matters, please feel free to continue to call upon Tom Horner of this office.

Daniel A. Craig

Daniel A. Craig
DISTRICT RANGER

INUNDATION STUDY

APPENDIX E

SANTA ANA RIVER PROJECTS

SEVEN OAKS INUNDATION STUDIES

FULL HEIGHT FAILURE ANALYSES

4 November 1987

CENPP-EN-HY
Portland District
Portland, Oregon

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CENPP-EN-HY
4 November 1987
Revised: 14 January 1988

I. SEVEN OAKS PROJECT DESIGN MODIFICATION

A. REFERENCE REPORT

This short report, Full Height Failure Analyses, is an update to the Summary Report which covered the Seven Oaks Inundation studies performed by the Portland District. The original Summary Report, as revised, is dated 29 October 1987. Information in that report was to be included in a General Design Memorandum.

The information in this report, Full Height Failure Analyses, covers only material different than that provided in the Summary Report. Except as provided herein, the Summary Report is applicable to these analyses of the revised Seven Oaks Dam design.

B. DESIGN MODIFICATION

Subsequent to the completion of studies described in the previous Summary Report (for the GDM), the design of Seven Oaks Dam was altered. The major change which affects the inundation studies is the reduction of the dam height. A corresponding lowering of the spillway crest changed both the full pool elevation and the maximum pool elevation. Top of dam elevation is 2609.4 feet, NGVD. The completed Seven Oaks Dam will be about 510 feet high. Full pool (spillway crest) is now located at elevation 2580 feet, NGVD, and maximum pool is at elevation 2604.4 feet. The total storage at the spillway crest is about 145,600 acre-feet.

C. INUNDATION STUDIES

As with the original design, two failure analyses were developed for the revised embankment. One assumed failure at full pool, elevation 2580 feet, NGVD (spillway crest). The second assumed failure at maximum pool, elevation 2604.4 feet, NGVD. Maximum pool is the highest level reached by the routed Probable Maximum Flood.

D. FULL HEIGHT FAILURE ANALYSES

In general, the information provided in the Summary Report covering earlier inundation studies is applicable to these two new analyses. Specifically, this would include information concerning the Computer Program; breach, channel routing, limitations, and flow regimes; the Study Reach; and Channel Geometry.

1. **Computer Program**

The dam failure analyses were performed using the National Weather Service DAMBRK program as before. Failure by overtopping was assumed for both analyses.

2. **Study Reach**

The reach for these inundation studies is the same as before; from Seven Oaks Dam to Prado Pool. This is a distance of about 33 miles.

3. **Channel Geometry and Roughness**

The same cross-sectional information was used for these studies. Roughness values were not changed significantly.

4. **Mapping**

Two separate sets of flooded area mapping will be developed, one for each analysis. These maps will include overlays which will allow color reproduction of the mapping with the flooded areas shown in color. The mapping is to be included in a future Feature Design Memorandum which will probably also contain Flood Emergency Planning information developed by the Los Angeles District.

II. FAILURE FOR COMPLETED SEVEN OAKS STRUCTURE

A. FAILURE at FULL POOL (Spillway Crest, elev. 2580')

1. **Breach Formation**

As described in the original Summary Report, two types of breach failures may be simulated using DAMBRK. These are failure by overtopping or failure by piping. Simulation of a failure of Seven Oaks dam with a static pool at the spillway crest would trigger the piping failure computations in DAMBRK. However, computational difficulties with the piping method were experienced in previous inundation studies. As the goal of this portion of DAMBRK is only to generate an outflow hydrograph, it not essential that the "actual" mode of failure be determined. All that is needed for the inundation study is to develop a breach hydrograph which may be routed downstream of the project. The requirement for the inundation study is to use a breach hydrograph with a peak outflow discharge which approximates that on the envelope curve of experienced dam failures (Summary Report, Figure 1). The volume of the hydrograph is that volume of water stored in the reservoir at time of failure.

A trapezoidal breach for the embankment was formed by failure of the completed structure at elevation 2580 feet, NGVD. For this analysis the top of dam was assumed to be at the spillway crest, elevation 2580 feet, NGVD. By using the spillway crest elevation, 2580', as "top of dam" elevation, DAMBRK will use the overtopping failure mode to generate an outflow hydrograph and model stability is increased. Breach parameters were adjusted to produce the peak discharge as described above. The bottom elevation of the breach is 2100 feet NGVD. Time for complete formation of the breach is 0.55 hours. The breach has a maximum bottom width of 150' and side slope of 0.75 H to 1.0 V. Inflow to the project during the failure analysis was a constant 1000 cfs. The volume of storage at this stage is 145,600 acre-feet.

2. Breach Hydrograph

The peak outflow resulting from this breach is 5,070,000 cfs. Sequential channel routing was employed. The routing reach was broken into a supercritical reach and a subcritical reach about 7 miles downstream of the project. The hydrograph attenuates to a peak discharge of 842,000 cfs at Prado pool.

3. Summary of Results

The hypothetical failure of Seven Oaks Project at full pool would cause considerable flooding downstream of the project. The flood wave would be on the order of 75 feet high through the canyon to Greenspot Road. Velocities as high as 160 feet per second would be experienced in the canyon area immediately below the project. At this point, where the river widens onto Santa Ana Wash, the general progression of the flood wave would probably be directed mainly along the left side of the wash. Although the effective flow region would be along the left bank, flooding would likely occur across the entire wash. Depths of up to 30-35 feet could be expected. Flow velocities would be high, ranging from 60 to 70 feet per second at the upstream end of the wash to about 30-35 feet per second opposite the Highland area. The Norton Air Force Base area and south San Bernardino would be flooded. Arrival time of the flood waters would be 45-50 minutes following the start of the breach. Parts of Colton would also be flooded. Velocities are generally in the 15 to 30 feet per second range. Maximum depths are typically 25 to 40 feet in the channelized river below the wash. The Rubidoux area, including Flabob Airport, would be flooded to the hills on the north. Downstream of Rubidoux, the Santa Ana flood plain narrows. The flood waters are more confined between the relatively high hills on either side. The flood plain area is relatively less developed than upstream. The flood wave entering Prado Pool area would have a peak discharge of about 842,000 cfs.

B. FAILURE at MAXIMUM POOL (PMF inflow, elev. 2604')

1. Breach Formation

A trapezoidal breach for the embankment was formed by failure of the completed structure at elevation 2604.4 feet, NGVD. For reasons stated in paragraph II.A.1., above, the top of structure was assumed to be at the maximum pool elevation, 2604.4 feet, NGVD. This assumption produces an outflow hydrograph which gives more reliable estimates of flood wave travel times than using the actual top of dam elevation, 2609.4 feet, NGVD. This is due to the computational scheme used by the program. Eroding a breach through the structure by piping, as would be the case if the top of dam were at 2609.4, does not materially affect the outflow hydrograph. By using elevation 2604.4 as the top of dam elevation, model stability is increased. The bottom elevation of the breach is 2100 feet NGVD. Time for complete formation of the breach is 0.55 hours. The breach has a maximum bottom width of 150' and side slope of 0.75 H to 1.0 V. Inflow to the project during the failure analysis was the PMF hydrograph. Maximum PMF inflow was 180,000 cfs. The volume of storage at this stage is 166,000 acre-feet.

2. Breach Hydrograph

The peak outflow resulting from this breach is 6,030,000 cfs. Sequential channel routing was employed. The routing reach was broken into a supercritical reach and a subcritical reach about 7 miles

downstream of the project. The hydrograph attenuates to a peak discharge of 1,080,000 cfs at Prado pool. Velocities are generally in the 15 to 30 feet per second range. Higher velocities, as high as 165 feet per second, are experienced in the canyon area immediately below the project. Maximum depths are typically 30 to 45 feet in the channelized river below the wash.

3. Summary of Results

The hypothetical failure of Seven Oaks Project at maximum pool would cause considerable flooding downstream of the project. The flood wave would be on the order of 80 feet high through the canyon to Greenspot Road. Velocities as high as 170 feet per second would be experienced in the canyon area immediately below the project. At this point, where the river widens onto Santa Ana Wash, the general progression of the flood wave would probably be directed mainly along the left side of the wash. Although the effective flow region would be along the left bank, flooding would likely occur across the entire wash. Depths of up to 30-35 feet could be expected. Flow velocities would be high, ranging from 60 to 70 feet per second at the upstream end of the wash to about 30-35 feet per second opposite the Highland area. The Norton Air Force Base area and south San Bernardino would be flooded. Arrival time of the flood waters would be 45-50 minutes following the start of the breach. Parts of Colton would also be flooded. Velocities are generally in the 15 to 30 feet per second range. Maximum depths are typically 25 to 40 feet in the channelized river below the wash. The Rubidoux area, including Flabob Airport, would be flooded to the hills on the north. Downstream of Rubidoux, the Santa Ana flood plain narrows. The flood waters are more confined between the relatively high hills on either side. The flood plain area is relatively less developed than upstream. The flood wave entering Prado Pool area would have a peak discharge of about 1,080,000 cfs.

III. SUMMARY TABLES

Tables 1 and 2 are summaries of breach assumptions and routed peak discharges, respectively, for the Seven Oaks Inundation Study. Table 3 is a summary of the results of the inundation studies for the full height Seven Oaks Project.

TABLE 1. Summary of Breach Parameters

Description	Bottom Width (Feet)	Time to Failure (Hours)	Side Slope (Z) *	Pool Storage (AF)	Failure Elevation (NGVD)	Hydraulic Depth (Feet)
Full Pool	150	0.55	0.75	145,600	2580.0	480
Maximum Pool	150	0.55	0.75	169,900	2604.4	504

* - (Z) Horizontal to 1.0 Vertical

TABLE 2. Summary of Peak Discharges at Selected Locations

Description	Peak Inflow Seven Oaks (cfs)	ROUTED PEAK DISCHARGE (cfs) Seven Oaks RM 0.0	Norton AFB RM 8.8	City of Riverside RM 21.1	Prado Pool RM 32.9
Full Pool	1,000	5,070,000	2,780,000	938,000	842,000
Maximum Pool	180,000	6,030,000	3,460,000	1,230,000	1,080,000

RM - River Mile measured downstream from Seven Oaks Project

TABLE 3. Summary of Results for Full Height Dam

Section Number		Distance from Dam (Miles)	Invert Elevation (feet, NGVD)	Arrival Time (Hours)	Peak Elevation (feet, NGVD)	Time of Peak Elevation (Hours)	Overbank Depth (feet)
Dam	F	0	2100	-	-	-	-
	M			-	-	-	-
5	F	0.7	1950	0.1	2023	0.5	63
	M			0.1	2029	0.5	69
10	F	3.8	1490	0.2	1532	0.6	32
	M			0.2	1535	0.6	35
13	F	7.0	1218	0.6	1261	0.8	36
	M			0.5	1264	0.8	39
15	F	8.8	1112	0.8	1151	1.2	26
	M			0.6	1155	1.0	30
20	F	12.9	959	1.5	1007	2.2	23
	M			1.3	1010	2.0	26
23	F	16.7	861	1.9	896	2.7	26
	M			1.7	901	2.5	31
26	F	21.1	758	2.6	789	3.4	32
	M			2.4	792	3.1	35
28	F	23.7	698	2.9	739	3.8	29
	M			2.8	744	3.7	34
30	F	26.7	646	3.2	674	4.0	21
	M			3.0	677	3.7	24
33	F	30.0	559	3.5	593	4.3	23
	M			3.3	597	4.0	27
35	F	32.9	514	3.9	545	4.6	20
	M			3.6	548	4.3	23

F - Failure at FULL POOL, elevation 2580 feet, NGVD.
M - Failure at MAXIMUM POOL, elevation 2604.4 feet, NGVD.

SANTA ANA RIVER PROJECTS

SEVEN OAKS INUNDATION STUDIES

REVISED SUMMARY REPORT

29 October 1987

CENPP-EN-HY
Portland District
Portland, Oregon

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SEVEN OAKS INUNDATION STUDY
CENPP-EN-HY
4 AUGUST 1987

Last Revised: 29 October 1987

I. GENERAL

The Seven Oaks Inundation Study is one work segment in a Memorandum of Understanding (MOU) between the Los Angeles District (SPL) and the Portland District (NPP). Other work to be performed by NPP under this MOU is Outlet Works design for both Seven Oaks and for Prado dams. Portland District Hydrology Section was the responsible for the Inundation Study.

The main objective of the dambreak analyses is to develop a breach hydrograph and route it downstream, developing peak stages and travel times in the process, and finally producing a flooded area boundary map. This information will be used to complete Emergency Plans for the affected areas downstream of the project. Because of this, parameters are chosen so as to produce "worst case" or, at least, conservatively high flood stages. It is better to be prepared for the worst and experience less than vice versa.

Table 1, below, compares the results of each inundation analysis by river location.

A. SCOPE/COST

The Seven Oaks Inundation Study involves several coordinated hydraulic computations. These are; to simulate the failure of a Seven Oaks Dam, compute the resultant outflow hydrograph, and simulate the movement of the dambreak flood through the downstream valley. The results of these computations can be used to develop potential inundation maps for the hypothetical failures and to establish times of travel of the flood wave to downstream locations. The dambreak study includes failure analyses for several different hypothetical dam failure modes. Conditions studied were for the cofferdam, for the main dam at various stages of construction, and for the completed structure at full pool and at maximum pool.

The inundation study scope of work includes eight specific conditions to be studied. These are: determination of a threshold cofferdam height; failure analysis for the final height cofferdam; failure analyses for four dam heights during construction; and failure analyses for the final dam height at full pool and at maximum pool. Four final sets of inundation mapping will be provided. Set one contains flood boundaries for the final height cofferdam. A second shows flood boundaries for the four construction

TABLE 1.
SUMMARY OF DAM-BREAK RESULTS by RIVER LOCATION

Section Number	DAM FAILURE HEIGHT (Feet)	Plate Number	Distance from Dam (Miles)	Invert Elevation (Feet, NGVD)	Arrival Time (Hours)	Peak Elevation (Feet, NGVD)	Time of Peak Elevation (Hours)	Average Overbank Depth (Feet)	Depth (Feet)
DAM	30' COFFERDAM	1	N/A	2100	N/A	2119	0.07	N/A	19
	100				N/A	2135	0.2	N/A	35
	200				N/A	2160	0.4	N/A	60
	300				N/A	2176	0.4	N/A	76
	400				N/A	2212	0.5	N/A	112
	SPILLWAY, 2598				N/A	2238	0.5	N/A	138
	PMF 2621.2				N/A	2250	0.5	N/A	150
5	30' COFFERDAM	1	0.7	1950	0.1	1961	0.1	1	11
	100				0.1	1973	0.2	13	23
	200				0.1	1994	0.4	34	44
	300				0.1	2009	0.4	49	59
	400				0.1	2014	0.5	54	64
	SPILLWAY, 2598				0.1	2026	0.5	66	76
	PMF 2621.2				0.1	2033	0.5	73	83
10	30' COFFERDAM	1	3.8	1490	0.3	1499	0.5	0	9
	100				0.4	1503	0.7	3	13
	200				0.4	1512	0.6	12	22
	300				0.3	1519	0.6	19	29
	400				0.3	1527	0.6	27	37
	SPILLWAY, 2598				0.3	1534	0.6	34	44
	PMF 2621.2				0.3	1536	0.6	36	46
13	30' COFFERDAM	1	7.0	1218	2.7	1223	2.9	0	5
	100				1.2	1227	1.4	1	9
	200				1.1	1239	1.4	15	21
	300				0.8	1246	1.0	20	28
	400				0.6	1253	0.9	27	35
	SPILLWAY, 2598				0.6	1262	0.8	36	44
	PMF 2621.2				0.6	1265	0.8	39	47
15	30' COFFERDAM	2	8.8	1112	3.5	1117	3.7	0	5
	100				1.3	1121	1.8	0	9
	200				1.3	1127	1.6	3	15
	300				1.0	1134	1.3	8	22
	400				0.8	1143	1.2	14	31
	SPILLWAY, 2598				0.8	1152	1.1	23	40
	PMF 2621.2				0.8	1156	1.0	27	44
20	30' COFFERDAM	2	12.9	959	5.4	965	6.2	0	6
	100				2.2	968	2.9	0	9
	200				1.9	976	3.0	0	17
	300				1.7	982	2.9	4	23
	400				1.6	1000	2.6	16	41
	SPILLWAY, 2598				1.5	1008	2.1	24	49
	PMF 2621.2				1.6	1011	1.9	27	52

TABLE 1. (continued)
SUMMARY OF DAM-BREAK RESULTS by RIVER LOCATION

Section Number	DAM FAILURE HEIGHT (Feet)	Plate Number	Distance from Dam (Miles)	Invert Elevation (Feet, NGVD)	Arrival Time (Hours)	Peak Elevation (Feet, NGVD)	Time of Peak Elevation (Hours)	Average Overbank Depth (Feet)	Depth (Feet)
23	30' COFFERDAM	3	16.7	861	6.5	868	7.5	0	7
	100				2.9	870	3.4	1	9
	200				2.5	876	3.4	6	15
	300				2.2	881	3.3	11	20
	400				2.2	889	3.1	19	28
	SPILLWAY, 2598				1.9	898	2.6	28	37
	PMF 2621.2				2.1	902	2.4	32	41
26	30' COFFERDAM	4	21.1	758	7.2	764	8.3	0	6
	100				3.4	766	4.0	0	8
	200				3.0	772	3.8	0	14
	300				2.7	777	4.0	17	19
	400				2.5	784	3.9	24	26
	SPILLWAY, 2598				2.3	790	3.3	30	32
	PMF 2621.2				2.5	793	3.1	33	35
28	30' COFFERDAM	4	23.7	698	7.8	705	9.0	0	7
	100				3.9	707	4.4	1	9
	200				3.4	715	4.1	5	17
	300				2.9	721	4.5	11	23
	400				2.8	731	4.3	21	33
	SPILLWAY, 2598				2.8	741	3.7	31	43
	PMF 2621.2				3.3	745	3.5	35	47
30	30' COFFERDAM	5	26.7	646	8.7	651	9.9	0	5
	100				4.4	653	4.9	1	7
	200				3.8	658	4.5	6	12
	300				3.3	662	4.7	9	16
	400				3.2	668	4.6	14	22
	SPILLWAY, 2598				3.1	675	3.9	21	29
	PMF 2621.2				3.8	678	3.7	24	32
33	30' COFFERDAM	5	30.0	559	9.8	563	11.0	0	4
	100				5.2	567	5.7	1	8
	200				4.4	575	5.2	6	16
	300				3.8	579	5.2	10	20
	400				3.8	586	4.9	16	27
	SPILLWAY, 2598				3.4	594	4.2	24	35
	PMF 2621.2				4.2	598	4.0	28	39
35	30' COFFERDAM	6	32.9	514	10.7	517	12.0	0	3
	100				5.8	521	6.4	3	7
	200				4.9	527	5.7	6	13
	300				4.1	532	5.5	9	18
	400				4.3	539	5.2	13	25
	SPILLWAY, 2598				3.9	546	4.5	20	32
	PMF 2621.2				4.6	549	4.2	23	35

interval heights and the full pool analyses. These two sets will be suitable for black and white reproduction. Two more sets of mapping to be provided will include color overlays of the flood boundaries. These are for the full pool failure and maximum pool failure analyses. This writeup for the Phase II GDM covering the inundation study work will be provided to SPL. Total estimated costs for the study were $73,610. A copy of the Memorandum of Understanding is attached. The Inundation Study description is contained in Appendix B of the MOU.

All information concerning the Seven Oaks Project; storage-elevation data, probable maximum flood, dam height, etc., were provided by Los Angeles District.

B. COMPUTER PROGRAM

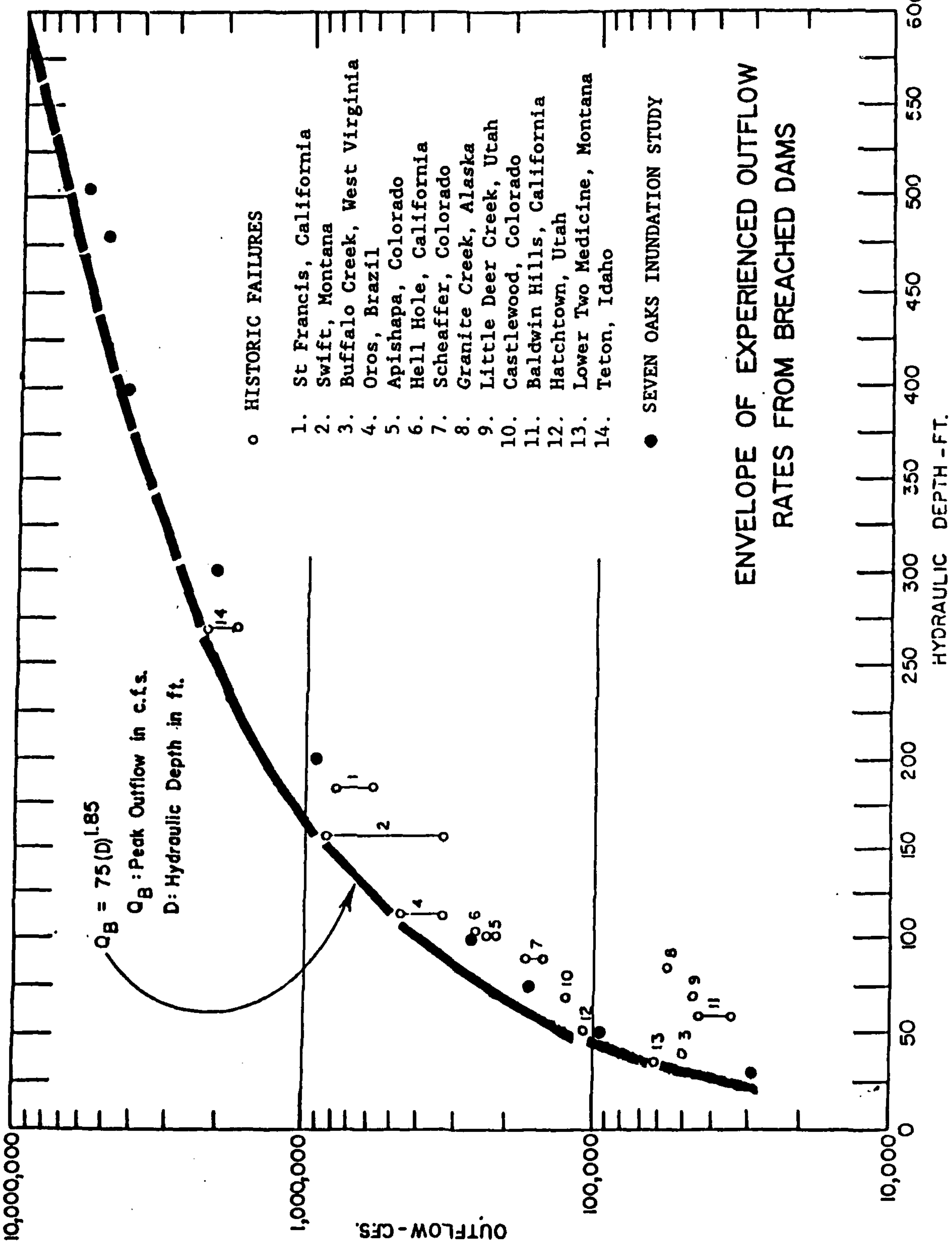
The dam failure analyses were performed using the July 1984 version of the DAMBRK computer program. This program was developed by the National Weather Service (NWS). The October 1986 version of the Hydrologic Engineering Center's (HEC) dambreak program preprocessor was also used as an input aid. The preprocessor program converts HEC style input to the NWS input format required by DAMBRK. Both programs were used on the NPP Harris 500 minicomputer.

1. Breach

Two types of breach failures may be simulated using DAMBRK, failure by overtopping or by piping. These two breach types form breach hydrographs of similar appearance. A trapezoidal failure breach was chosen for this study. Parameters which affect the breach formation, and hence the outflow hydrograph, are; the pool level in the reservoir at time of failure, the time for full formation of the breach, and the shape and maximum size of the breach. These parameters were manipulated within Corps of Engineers guidelines to produce an outflow hydrograph with peak discharge approaching, but less than, that shown in an envelope curve of historical dam failures. (Figure 1). All breaches were assumed to erode to the channel invert, elevation 2100 feet NGVD.

2. Channel Routing

The river routing procedure is based on solving the one-dimensional equations of motion for unsteady flow in an open channel. The breach outflow hydrograph was routed through the downstream channel using dynamic wave hydraulic routing methods which include the complete Saint Venant flow equations. The components of the flow equations include terms for; local acceleration, convective acceleration, pressure gradient, friction force, and gravity. By using the full equations, the model is able to account for the acceleration effects associated with dambreak hydrographs and account for the influence of downstream unsteady backwater effects. These backwater effects can be produced by channel constrictions, bridges, road embankments, etc.



Reference: FLOOD EMERGENCY PLANS; Guidelines for Corps Dams; Hydrologic Engineering Center; Davis, California; June 1980.

Figure 1.

3. Limitations

The channel routing method does have several limitations. It is a one-dimensional analysis. This implies that cross sections are oriented perpendicular to the flow so that the water surface is horizontal across the section. Superelevation and secondary currents are not considered. Situations that involve large off-channel storage areas that take a long time to fill and empty may not be accurately simulated. The assumption is made that the channel boundaries are rigid, that is, cross sections do not change shape due to scour or deposition. The program uses a simplistic cross sectional geometry of elevation versus topwidth. The routing must be started with an initial channel condition (wet channel) downstream of the project. Changes from subcritical flow to supercritical flow with either time or distance cannot be computed. The reach may be divided into two subreaches if desired; the upper subreach being entirely supercritical and the lower being subcritical. This technique, called "sequential" routing, was used for the various failure analyses.

4. Flow Regime

As discussed in the "Limitations" Section, changes from supercritical flow to subcritical flow cannot be computed by DAMBRK. The reach may, however, be designated by the user as either supercritical or subcritical. Whichever regime is chosen, it is used for all levels of flow in the reach. DAMBRK documentation suggest a change from supercritical to subcritical flow at a channel slope of about 0.0095 (50 feet/mile). This would, of course, depend on the channel configuration. The 0.0095 slope was used as a starting point in dividing the Santa Ana River into supercritical and subcritical reaches. Several iterations of were run shifting this location, based on computed Froude numbers, until a reasonable compromise was found and nonconvergence problems with the program were eliminated or minimized. The supercritical/subcritical division was not necessarily the same for all of the different dam heights studied. The specific location for each dam height is described in the appropriate section of the report.

II. STUDY REACH DESCRIPTION

A. STUDY REACH

The study reach for this project was from the Seven Oaks Dam on the upper Santa Ana River to the Prado Dam pool approximately 33 miles downstream.

Stationing for the channel routing portion of DAMBRK is in units of river miles, measured downstream from the dam. The distances were taken from the United States Geological Survey (USGS) 7 1/2 minute quadrangle maps described in Section II E, below. The distances used are generally compromises between straight line valley distances and channel measurements (following all meanders). More weight was given to the valley distances as magnitude of most of the failure hydrographs will result in considerable overbank flow.

B. SEVEN OAKS PROJECT

Seven Oaks Project is to be located about eleven miles upstream of San Bernardino on the Santa Ana River. It will be a high (over 500') earth embankment designed primarily for flood control purposes. Total storage at the spillway crest, elevation 2598 feet, NGVD, will be 160,000 acre-feet. Table 2 shows reservoir storage at each of the assumed failure heights.

TABLE 2. Reservoir Storage at Dam Failure Elevations

Embankment Elevation (Feet, NGVD)	Nominal Height (Feet)	Storage Capacity (Acre-Feet)
2100 (Invert)	0	0
2130	30	163
2150	50	552
2175	75	1,460
2200	100	2,970
2300	200	16,300
2400	300	43,300
2500	400	90,400
2598 (Full Pool)	498	160,000
2621 (Max. Pool)	520	181,000

C. SEVEN OAKS PROJECT DESIGN MODIFICATION

Subsequent to the completion of studies described in this report, the design of Seven Oaks Dam was altered. The major change which affects this study is the reduction of the dam height. A corresponding lowering of the spillway crest changed both the full pool elevation and the maximum pool elevation. Full pool (spillway crest) is now located at elevation 2580 feet, NGVD, and maximum pool is at elevation 2604.4 feet. Additional inundation studies and mapping will completed to reflect these changes. Study results for the revised project height will be provided for inclusion in a later Design Memorandum.

D. CHANNEL DESCRIPTION

The Santa Ana River characteristics change considerably through the study reach. It flows generally southwest from Seven Oaks Dam to Prado Dam. Along the way it passes though numerous communities. At the upstream end, near Seven Oaks Dam, the Santa Ana River is an extremely steep canyon stream. The bed between the canyon walls is composed of large boulders on the surface overlaying smaller sized coarse material. Approximately one and a third miles downstream of the dam, the stream opens up onto an alluvial fan, the Santa Ana Wash. This wash extends to the upstream side of San Bernardino, about seven and a half miles. The bed material is coarse, typically decreasing in size in the downstream direction. The wash is fairly wide, up to a mile and a half, and is still fairly steep. The slope

of the wash ranges from about 0.0322 (170 feet/mile) at the canyon mouth to near 0.0095 (50 feet/mile) at the downstream end near San Bernardino. It is bounded by high hills on the north side and by a fairly well defined bank on the south. Beginning at San Bernardino the Santa Ana River returns to a "river" configuration. It is well channelized by natural and manmade embankments. The channel slope varies from 0.0095 (50 feet/mile) at San Bernardino to about 0.0028 (15 feet/mile) at Prado Dam pool. From San Bernardino to Prado Dam the channel is generally clear of vegetation with a few exceptions. The major concentration of channel vegetation begins near Riverside.

E. CHANNEL GEOMETRY

USGS 7 1/2' quadrangle maps, dated 1967 (1980 photorevision) were used for developing input cross sections for the computer program. These maps were also used to develop base maps for mapping of flooded area boundaries. The configuration of the Santa Ana River, especially the wash near Redlands, appears to have changed considerably since the quad maps were issued. The flood of 1969 and major channel modification work have both occurred since the last major revision of the USGS maps. The contour interval for these maps is generally 20 feet. Topographic mapping near the project is in 40 foot intervals. Precision of both mapping and cross sections is therefore generally no better than plus or minus 10 feet or plus or minus 20 feet near the project. Based on this mapping, the computed water surface elevations for the inundation analyses would have this same precision, plus or minus 10 to 20 feet.

The "accuracy" of the computed water depths at a river location are dependent upon several factors. The defined accuracy would be that of the least accurate information used in developing the model. This again relates to the available mapping. The primary factor affecting computed depth would be the conveyance which is described by the channel cross-sectional information input to the model. This accuracy of depth would be better than that for the computed ground elevations discussed above. Although there is not a good way to quantify the depth accuracy, subjectively, it would be on the order of plus or minus 3 to 5 feet. This is assuming that the relative topography on the quad maps is more accurate than the absolute elevation shown. That is, using the quad maps the cross-section shape can be which is described better than it can be actually referenced to real ground elevation based on the maps.

Thirty-five cross-sections were developed from the USGS mapping and used in the river model. The DAMBRK program developed additional cross-sections to use in its computation scheme by interpolating between the input cross-sections. Approximately 90 interpolated cross-sections were created for a total of 125 to be used in the channel routing. Spacing between the cross-sections varied from about 0.05 miles (260 feet) to 0.5 miles. This is the spacing including interpolated sections. Closer spacing is required in the areas of higher velocities. Therefore it is used the canyon area immediately below the project. The wider spacing is in the channel reach from about Riverside to Prado pool.

Rather than the customary HEC-2 elevation/station format, the cross sectional input to the DAMBRK program is in the form elevation/active topwidth/storage topwidth. There is a limit of eight of these sets per cross section.

F. CHANNEL ROUGHNESS

The Santa Ana River channel roughness changes from higher to lower roughness in the downstream direction. The Manning's "n" values were also varied by depth of flow at a location. Near Seven Oaks Dam Manning's "n" values ranged from 0.100 to 0.070. Values near 0.035 were used for the cleaner stream areas near the downstream end of the study reach. Roughness values were generally estimated using experience in steep Northwest streams, the USGS Water Supply Paper 1849, Roughness Characteristics of Natural Channels, and information provided by SPL.

Manning's roughness values were used to reduce nonconvergence problems in the DAMBRK computational scheme. Varying these values (along with interpolated cross sections, channel smoothing, and expansion and contraction coeffcients) augmented the program's automatic convergence procedures. Table 3 lists the "n" value ranges along the Santa Ana River as used in the DAMBRK models.

TABLE 3. Manning's Roughness Values (n)

River Mile to	River Mile	"n" value Range
0	1	0.050 - 0.100
1	2	0.050 - 0.070
2	3	0.048 - 0.065
3	4	0.048 - 0.060
4	6	0.045 - 0.055
6	10	0.040 - 0.050
10	33	0.035 - 0.045

III. THRESHOLD COFFERDAM FAILURE

A. THRESHOLD COFFERDAM

The "Threshold" cofferdam was which is described as that height of cofferdam which, if failed catastrophically, would cause relatively minimal damages to overbank areas downstream of the project. The damage reach assumed for this analysis was from Seven Oaks Project downstream to the interchange of Highways 10 and 215 at San Bernardino. The interchange is about 13 miles downstream of Seven Oaks damsite. The results of this analysis should be used only as a guide to possible risk, not as precise design criteria.

Two "threshold" cofferdam heights were actually developed. One cofferdam "threshold" height is the height of cofferdam which would not cause damage

to overbank areas immediately downstream of the project; in particular to Powerhouse Number 3 about one mile downstream of the project site. This criterion severely limited the threshold height of the cofferdam. The height of cofferdam based on this criterion was determined to be 30 feet high.

A second, and higher, cofferdam "threshold" height was developed which would not cause substantial damage to areas further downstream of the project. In this second case flooding of the powerhouse was not considered as "substantial" damage. Failure of a cofferdam of this given height would flood Powerhouse 3, one mile downstream of the project site, but would not cause major flooding of downstream communities. This second height was determined to be 75 feet high.

B. 30 FOOT COFFERDAM

The 30 foot cofferdam was sized so as to avoid damaging the powerhouse downstream of the project. Time for complete formation of the breach is 0.07 hours. The breach has a maximum bottom width of 50 feet and a side slope of 0.5 horizontal (H) to 1.0 vertical (V). Peak breach outflow from this cofferdam failure would be about 28,100 cfs. The routing reach was divided into a supercritical flow region and a subcritical flow region at a point 0.18 miles downstream of the cofferdam. Manning's "n" values ranged from 0.100 near the project to 0.035 at San Bernardino. Maximum stages range from 10 to 12 feet in the upper canyon to 5 to 6 feet downstream. Maximum flow velocities were about 15 feet per second in the canyon and 4 or 5 feet per second in the downstream channel near San Bernardino.

The flood could do some damage to bridge abutments and exposed foundations along the river, but no major damage outside the channel would be expected. The tailrace at the powerhouse would probably be damaged to some extent. The main danger resulting from failure of a 30 foot high cofferdam would be to persons and equipment located in the channel of the Santa Ana River or on low bridges (near the powerhouse) or on highwater overflow roads (e.g. Alabama Street/Palm Avenue).

C. 75 FOOT COFFERDAM

Failure of the 75 foot cofferdam would produce a peak outflow of 157,000 cfs. Time for complete formation of the breach is 0.40 hours. The breach has a maximum bottom width of 150 feet and a side slope of 1.0 H to 1.0 V. Flooding would extend from valley wall to valley wall within the canyon area upstream of Greenspot Road. The floodwave would reach a maximum height of about 20 feet through the canyon with velocities in the 30 to 35 feet/second range. Substantial erosion of bed material and unprotected fill material could occur. Construction activities located along the river could be affected. Downstream of the canyon mouth, the breach hydrograph would attenuate rapidly. The flow would generally be confined to the channel along the left bank of the wash until it reached the relatively well-which is described channel near the Municipal Airport about four miles downstream of the project site. The flood waters would follow low gulleys, ditches, and ephemeral streams as they proceed downstream. The hydrograph would be contained well "within banks" by the time it reached the city of San

Bernardino. Channel erosion and damage to bridges and bridge abutments would be likely.

Failure of a cofferdam higher than 75 feet would result in overbank flow in the San Bernardino area. Norton Air Force Base specifically could be flooded to some extent, at least disrupting air traffic. Failure of higher structures would affect more and more downstream areas as the structure height increased.

D. THRESHOLD COFFERDAM RE-ANALYSIS

1. General

Following submittal of the two threshold heights described above, Los Angeles District indicated that the primary concern of the threshold cofferdam analyses should be keeping Powerhouse 3 out of harms way. In other words, to limit the cofferdam to that height which would not flood the powerhouse should the cofferdam fail. Based on the available USGS quadrangle mapping, that cofferdam would be 30 feet high, as described above.

However, new project mapping had just been completed in the canyon area of the Santa Ana River downstream of Seven Oaks Project site. This mapping is to a scale of 1"=40' with as small as 1 foot contour intervals in the channel area. The new topographic mapping sent to NPP covered the Santa Ana River from the project to just downstream of Powerhouse 3. The study reach for this new analysis was limited to this channel reach. Using this new mapping, the threshold cofferdam was re-analyzed to determine the safe cofferdam height for Powerhouse 3.

2. Results

The analysis shows that the threshold cofferdam could be 50 feet in height. The hydrograph formed by failure of this cofferdam would not reach the level of Powerhouse Number 3 located 1 mile downstream of the project.

Failure of a dam 50 feet high would release a peak discharge of about 98,000 cfs. Time for complete formation of the breach is 0.06 hours. The breach has a maximum bottom width of 80 feet and a side slope of 0.5 H to 1.0 V. The floodwave peak would reach the Powerhouse location about 0.07 hours (4-5 minutes) later. Peak discharge at the Powerhouse is 72,000 cfs. The "freeboard" for peak stage at the Powerhouse is about 1/2 foot. Maximum velocities were in the 15 to 20 feet per second range.

Based on this re-analysis, Los Angeles District has decided to increase the cofferdam height to 50 feet from 30 feet. Table 4 contains estimates of arrival times, peak elevations, overbank depths, etc. for assumed failure of the 50-foot high structure. Model results are used from the cofferdam downstream to about Greenspot Road bridge. The remainder of the information, from Greenspot Road bridge to Prado pool, is based on interpolation of the 30 foot and 75 foot cofferdam analyses. The interpolated data should be entirely adequate for planning purposes, especially since the cofferdam is transitory.

TABLE 4. Summary of Information for 50 foot Cofferdam

Section Number	Distance from Dam (Miles)	Invert Elevation (feet, NGVD)	Arrival Time (Hours)	Time of Peak Elevation (feet, NGVD)	Time of Peak Elevation (Hours)	Overbank Depth (feet)
Dam	-	2100	-	-	-	-
6	1.0	1940	0.1	1960	0.1	0
10	3.8	1490	0.4	1500	0.5	0
13	7.0	1218	2.0	1224	2.1	0
15	8.8	1112	2.6	1118	2.7	0
20	12.9	959	4.0	966	4.5	0
23	16.7	861	5.0	868	5.6	0
26	21.1	758	5.5	764	6.2	0
28	23.7	698	5.9	705	6.8	0
30	26.7	646	6.7	652	7.6	0
33	30.0	559	7.5	565	8.6	0
35	32.9	514	8.9	519	10.0	0

3. Summary

The increase in threshold cofferdam height from 30 feet to 50 feet is based on a considerable difference in the topography used in the dam break analyses. The new mapping shows the channel in sufficiently greater detail to indicate an increase of 10 to 12 feet in usable channel conveyance (depth) in the area of the powerhouse. This increase in conveyance allows "safe" passage of a larger breach hydrograph.

The breach hydrograph is an estimate based generally on the envelope curve shown in FLOOD EMERGENCY PLANS, Guidelines for Corps Dams, Hydrologic Engineering Center. Actual cofferdam failure could produce a peak discharge greater or less than that shown by the curve. The flow regime in the area of Powerhouse 3 appears to be extremely complex and variable. As mentioned above, the NWS DAMBRK program uses a one dimensional computational scheme. Superelevation and secondary currents are not considered. Also, there are numerous unknown factors which could occur during a cofferdam failure. Should there be a considerable quantity of flow in the river before the failure, the powerhouse could be in danger from the addition of a failure floodwave. If the bridge by the gaging station directly across the river from the powerhouse were to remain in place and trap debris, flow could be diverted toward the powerhouse. Also, the conduit crossing the river at powerhouse could increase water surface levels should it remain in place and trap even minimal quantities of debris.

For these reasons, it is recommended that a cofferdam height not be chosen solely or primarily on information developed using a dam break analysis. These studies develop information which made be used to assess possible risk but contain too many assumptions to use as a sole basis for design.

IV. FULL HEIGHT COFFERDAM FAILURE (50' High)

The "full height" cofferdam is the cofferdam which will be used during construction of Seven Oaks dam. Responsibility for selection of the cofferdam height is with SPL. The height was originally selected to be 30 feet. It has recently been revised upward to 50 feet based on information described in Section III D, above.

Failure of an embankment of 50 feet could produce a peak outflow of about 98,100 cfs. Flow would generally be contained within banks downstream of Greenspot Road bridge. It is not likely that substantial damage would occur except for in-channel operations (Seven Oaks construction, downstream gravel operations, etc.). The main danger resulting from failure of a 50 foot high cofferdam would probably be to persons and equipment located in or near the Santa Ana River channel, on low bridges (near the powerhouse), or on highwater overflow roads (e.g. Alabama Street/Palm Avenue).

Manning's "n" values ranged from 0.100 near the project to 0.035 at San Bernardino. Maximum stages range from 10 to 12 feet in the upper canyon to 5 to 6 feet downstream. Maximum flow velocities were about 15-20 feet per second in the canyon and 4-5 feet per second in the downstream channel near San Bernardino.

V. DAM FAILURE DURING CONSTRUCTION OF SEVEN OAKS DAM

Four hypothetical failures were developed for partially completed stages of Seven Oaks Dam. The intervals picked were in 100 feet increments. The analyses will show flooding which could occur should the project fail during the construction period. No attempt was made to establish the construction schedule for these stages, only equal increments of embankment height.

A. FAILURE at CONSTRUCTION INTERVAL 100'

A trapezoidal breach for the 100' construction interval was formed by failure of the partially completed structure. Top of structure is at elevation 2200 feet, NGVD. The bottom elevation of the breach is 2100 feet NGVD. Time for complete formation of the breach is 0.23 hours. The breach has a maximum bottom width of 100' and side slope of 0.5 H to 1.0 V. Inflow to the project during the failure analysis was a constant 1000 cfs. The volume of storage at this stage is 2970 acre-feet.

The peak outflow resulting from this breach is 252,000 cfs. Sequential channel routing was employed. The routing reach was broken into a supercritical reach and a subcritical reach about 5 miles downstream of the project. The hydrograph attenuates to a peak discharge of 31,200 cfs at Prado pool. Velocities are generally in the 6 to 10 feet per second range. Higher velocities being in the steeper upstream areas of the study reach. Maximum depths are typically 8-9 feet in the channelized river below the wash.

B. FAILURE at CONSTRUCTION INTERVAL 200'

A trapezoidal breach for the 200' construction interval was formed by failure of the partially completed structure. Top of structure is at elevation 2300 feet, NGVD. The bottom elevation of the breach is 2100 feet NGVD. Time for complete formation of the breach is 0.25 hours. The breach has a maximum bottom width of 150' and side slope of 0.5 H to 1.0 V. Inflow to the project during the failure analysis was a constant 1000 cfs. The volume of storage at this stage is 16,300 acre-feet.

The peak outflow resulting from this breach is 918,000 cfs. Sequential channel routing was employed. The routing reach was broken into a supercritical reach and a subcritical reach about 5 miles downstream of the project. The hydrograph attenuates to a peak discharge of 131,000 cfs at Prado pool. Velocities are generally in the 8 to 20 feet per second range. Higher velocities, near 50 feet per second, are experienced in the canyon area immediately below the project. Maximum depths are typically 15 feet in the channelized river below the wash.

C. FAILURE at CONSTRUCTION INTERVAL 300'

A trapezoidal breach for the 300' construction interval was formed by failure of the partially completed structure. Top of structure is at elevation 2400 feet, NGVD. The bottom elevation of the breach is 2100 feet NGVD. Time for complete formation of the breach is 0.40 hours. The breach has a maximum bottom width of 150' and side slope of 0.5 H to 1.0 V. Inflow to the project during the failure analysis was a constant 1000 cfs. The volume of storage at this stage is 43,300 acre-feet.

The peak outflow resulting from this breach is 2,020,000 cfs. Sequential channel routing was employed. The routing reach was broken into a supercritical reach and a subcritical reach about 5 miles downstream of the project. The hydrograph attenuates to a peak discharge of 207,000 cfs at Prado pool. Velocities are generally in the 10 to 25 feet per second range. Higher velocities, as high as 90 feet per second, are experienced in the canyon area immediately below the project. Maximum depths are typically 20 feet in the channelized river below the wash.

D. FAILURE at CONSTRUCTION INTERVAL 400'

A trapezoidal breach for the 400' construction interval was formed by failure of the partially completed structure. Top of structure is at elevation 2500 feet, NGVD. The bottom elevation of the breach is 2100 feet NGVD. Time for complete formation of the breach is 0.45 hours. The breach has a maximum bottom width of 150' and side slope of 0.6 H to 1.0 V. Inflow to the project during the failure analysis was a constant 1000 cfs. The volume of storage at this stage is 90,400 acre-feet.

The peak outflow resulting from this breach is 3,710,000 cfs. Sequential channel routing was employed. The routing reach was broken into a supercritical reach and a subcritical reach about 7 miles downstream of the project. The hydrograph attenuates to a peak discharge of 519,000 cfs at Prado pool. Velocities are generally in the 12 to 25 feet per second range.

Higher velocities, as high as 120 feet per second, are experienced in the canyon area immediately below the project. Maximum depths are typically 25 to 30 feet in the channelized river below the wash.

VI. FAILURE FOR COMPLETED SEVEN OAKS STRUCTURE

A. GENERAL

The completed Seven Oaks Project will be about 560 feet high. Top of dam elevation is 2626.5 feet, NGVD. The spillway crest is located at elevation 2598 feet, NGVD. The total storage at the spillway crest elevation is about 160,000 acre-feet.

Two failure analyses were developed for the completed embankment. One assumed failure at full pool elevation (spillway crest). The second assumed failure at maximum pool, elevation 2621.2 feet, NGVD. Maximum pool is the elevation reached by the routed Probable Maximum Flood.

B. FAILURE at FULL POOL (Spillway Crest, elev. 2598')

1. Breach Formation

A trapezoidal breach for the embankment was formed by failure of the completed structure at elevation 2598 feet, NGVD. For this analysis the top of structure was assumed to be at the spillway crest, elevation 2598 feet, NGVD. This assumption produced a "cleaner" analysis and more reliable estimates of flood wave travel times than using the actual top of dam elevation, 2626.5 feet, NGVD. This is due to the computational scheme used by the program. Eroding a breach through the structure by piping, as would be the case if the top of dam were at 2626.5, does not materially affect the outflow hydrograph. By using elevation 2598 as the top of dam elevation, model stability is increased. The bottom elevation of the breach is 2100 feet NGVD. Time for complete formation of the breach is 0.55 hours. The breach has a maximum bottom width of 150' and side slope of 0.75 H to 1.0 V. Inflow to the project during the failure analysis was a constant 1000 cfs. The volume of storage at this stage is 160,000 acre-feet.

2. Breach Hydrograph

The peak outflow resulting from this breach is 5,570,000 cfs. Sequential channel routing was employed. The routing reach was broken into a supercritical reach and a subcritical reach about 7 miles downstream of the project. The hydrograph attenuates to a peak discharge of 914,000 cfs at Prado pool.

3. Summary of Results

The hypothetical failure of Seven Oaks Project at full pool would cause considerable flooding downstream of the project. The flood wave would be on the order of 75 feet high through the canyon to Greenspot Road. Velocities as high as 160 feet per second would be experienced in the canyon area immediately below the project. At this point, where the river widens onto

Santa Ana Wash, the general progression of the flood wave would probably be directed mainly along the left side of the wash. Although the effective flow region would be along the left bank, flooding would likely occur across the entire wash. Depths of up to 20-25 feet could be expected. Flow velocities would be high, ranging from 60 to 70 feet per second at the upstream end of the wash to about 30-35 feet per second opposite the Highland area. The Norton Air Force Base area and south San Bernardino would be flooded. Arrival time of the flood waters would be 45-50 minutes following the start of the breach. Parts of Colton would also be flooded. Velocities are generally in the 15 to 30 feet per second range. Maximum depths are typically 25 to 40 feet in the channelized river below the wash. The Rubidoux area, including Flabob Airport, would be flooded to the hills on the north. Downstream of Rubidoux, the Santa Ana flood plain narrows. The flood waters are more confined between the relatively high hills on either side. The flood plain area is relatively less developed than upstream. The flood wave entering Prado Pool area would have a peak discharge of about 914,000 cfs.

C. FAILURE at MAXIMUM POOL (PMF inflow, elev. 2621')

1. Breach Formation

A trapezoidal breach for the embankment was formed by failure of the completed structure at elevation 2621.2 feet, NGVD. For this analysis the top of structure was assumed to be at the maximum pool elevation, 2621.2 feet, NGVD. This assumption produced a "cleaner" analysis and more reliable estimates of flood wave travel times than using the actual top of dam elevation, 2626.5 feet, NGVD. This is due to the computational scheme used by the program. Eroding a breach through the structure by piping, as would be the case if the top of dam were at 2626.5, does not materially affect the outflow hydrograph. By using elevation 2621.2 as the top of dam elevation, model stability is increased. The bottom elevation of the breach is 2100 feet NGVD. Time for complete formation of the breach is 0.55 hours. The breach has a maximum bottom width of 150' and side slope of 0.75 H to 1.0 V. Inflow to the project during the failure analysis was the PMF hydrograph. Maximum PMF inflow was 180,000 cfs. The volume of storage at this stage is 181,000 acre-feet.

2. Breach Hydrograph

The peak outflow resulting from this breach is 6,530,000 cfs. Sequential channel routing was employed. The routing reach was broken into a supercritical reach and a subcritical reach about 7 miles downstream of the project. The hydrograph attenuates to a peak discharge of 1,160,000 cfs at Prado pool. Velocities are generally in the 15 to 30 feet per second range. Higher velocities, as high as 165 feet per second, are experienced in the canyon area immediately below the project. Maximum depths are typically 30 to 45 feet in the channelized river below the wash.

3. Summary of Results

The hypothetical failure of Seven Oaks Project at maximum pool would cause considerable flooding downstream of the project. The flood wave would be on

the order of 80 feet high through the canyon to Greenspot Road. Velocities as high as 170 feet per second would be experienced in the canyon area immediately below the project. At this point, where the river widens onto Santa Ana Wash, the general progression of the flood wave would probably be directed mainly along the left side of the wash. Although the effective flow region would be along the left bank, flooding would likely occur across the entire wash. Depths of up to 30-35 feet could be expected. Flow velocities would be high, ranging from 60 to 70 feet per second at the upstream end of the wash to about 30-35 feet per second opposite the Highland area. The Norton Air Force Base area and south San Bernardino would be flooded. Arrival time of the flood waters would be 45-50 minutes following the start of the breach. Parts of Colton would also be flooded. Velocities are generally in the 15 to 20 feet per second range. Maximum depths are typically 25 to 40 feet in the channelized river below the wash. The Rubidoux area, including Flabob Airport, would be flooded to the hills on the north. Downstream of Rubidoux, the Santa Ana flood plain narrows. The flood waters are more confined between the relatively high hills on either side. The flood plain area is relatively less developed than upstream. The flood wave entering Prado Pool area would have a peak discharge of about 1,160,000 cfs.

VII. SUMMARY TABLES

Tables 5 and 6, on the next page, are summaries of breach assumptions and routed peak discharges, respectively, for the Seven Oaks Inundation Study. The tables also include information for the revised Seven Oaks design (see Section II C).

TABLE 5. Summary of Breach Parameters

Description	Bottom Width (Feet)	Time to Failure (Hours)	Side Slope (Z) *	Pool Storage (AF)	Failure Elevation (NGVD)	Hydraulic Depth (Feet)
Threshold (75')	150	0.40	1.0	1,460	2175	75
30' Cofferdam	50	0.07	0.5	163	2130	30
50' Cofferdam	80	0.12	0.5	552	2150	50
Const. Interval 1	100	0.23	0.5	2,970	2200	100
Const. Interval 2	150	0.35	0.5	16,300	2300	200
Const. Interval 3	150	0.40	0.5	43,300	2400	300
Const. Interval 4	150	0.45	0.5	90,400	2500	400
Full Pool (Original)	150	0.55	0.75	160,400	2598.0	498
Maximum Pool (")	150	0.55	0.75	181,200	2620.2	520
Full Pool (Revised)	150	0.55	0.75	145,600	2580.0	480
Maximum Pool (")	150	0.55	0.75	169,900	2604.1	504

* - (Z) Horizontal to 1.0 Vertical

TABLE 6. Summary of Peak Discharges at selected location

Description	Peak Inflow Seven Oaks (cfs)	ROUTED PEAK DISCHARGE (cfs) Seven Oaks RM 0.0	Norton AFB RM 8.8	City of Riverside RM 21.1	Prado Pool RM 32.9
Threshold (75')	1,000	157,000	61,400	-	-
30' Cofferdam	1,000	28,100	5,850	4,330	4,220
50' Cofferdam	1,000	98,100	-	-	-
Const. Interval 1	1,000	252,000	67,300	42,100	31,200
Const. Interval 2	1,000	918,000	297,000	151,000	131,000
Const. Interval 3	1,000	2,040,000	792,000	278,000	267,000
Const. Interval 4	1,000	3,720,000	1,670,000	565,000	519,000
Full Pool (Original)	1,000	5,570,000	3,060,000	1,030,000	914,000
Maximum Pool (")	180,000	6,540,000	3,750,000	1,330,000	1,160,000
Full Pool (Revised)	1,000	5,070,000	2,780,000	938,000	842,000
Maximum Pool (")	180,000	6,030,000	3,460,000	1,230,000	1,080,000

RM - River Mile measured downstream from Seven Oaks Project

VIII. RESOURCES

Training Course Class Material, Analytical Techniques for Dam Break Analysis, Hydrologic Engineering Center, U S Army Corps of Engineers, January 1980

User's Manual, DAMBRK, The NWS Dam-break Flood Forecasting Model, Hydrologic Research Laboratory, National Weather Service, National Oceanic and Atmospheric Administration, February 1984; Documentation by Hydrologic Engineering Center, U S Army Corps of Engineers

DAMBRK: The NWS Dam-Break Flood Forecasting Model, D. L. Fread, Office of Hydrology, National Weather Service, July 1983

Flood Emergency Plans, Guidelines for Corps Dams, Hydrologic Engineering Center, U S Army Corps of Engineers, June 1980

Roughness Characteristics of Natural Channels, Water Supply Paper 1849, U S Geological Survey, 1967

F ASSESSMENT · Embankment and Roller Compacted Concrete Dams

SANTA ANA RIVER BASIN, CALIFORNIA

PHASE I ASSESSMENT

EMBANKMENT AND ROLLER COMPACTED CONCRETE DAMS

SEVEN OAKS DAM

31 DECEMBER 1986

DEPARTMENT OF THE ARMY
SOUTH PACIFIC DIVISION, CORPS OF ENGINEERS
630 Sansome Street, Room 720
San Francisco, California 94111-2206

REPLY TO
ATTENTION OF:

CESPD-ED (1130-2-305b)

MEMORANDUM FOR: CDR USACE (CEEC-E) 20 MASSACHUSETTS AVE., N.W., WASH DC
20314-1000 9 Jul 87

SUBJECT: Roller Compacted Concrete (RCC) Alternative for Seven Oaks Dam, Santa Ana, California

SUMMARY STATEMENT.

1. This letter is in response to your guidance on the degree to which an RCC design concept for the Seven Oaks site should be pursued during the development of the Santa Ana River Project GDM. We have roughly scoped the RCC design for the Seven Oaks dam site, visited the Portland District Elk Creek project, provided the proposed RCC design concept to the Portland District design staff, had outside technical consultants in geotechnical, structural, and seismicity fields review both the RCC and embankment concepts, and have come to the conclusion that RCC is not technically feasible under the design criteria considered appropriate for this site. The following paragraphs summarize the several issues which were considered by us in coming to this conclusion. The issues are:

a. the established design criteria of 4' foundation displacement, in any direction, resulting from an 8+ Richter Magnitude seismic event.

b. technical feasibility and "state-of-the-art" knowledge of the RCC concept and the earth and rockfill embankment design concept for a 550' high structure.

c. reservoir water surface elevation assumptions for structural stability analyses and the probability of joint occurrence of independent events (i.e., an earthquake and high water);

d. an order of magnitude cost comparison of the two concepts; and,

e. the practicability of carrying on parallel designs, even to a minor extent, for both concepts during development of the GDM.

All of these issues are addressed in detail in the attached material, referenced in paragraph 18, and which include our endorsement providing additional guidance to your letter requesting the preparation of a Phase I Technical Feasibility Report and a Phase II Cost Comparison Report; the Los Angeles District endorsement back to us summarizing their Phase I findings; the Los Angeles District Phase I Assessment Report; the Portland District technical opinions; and all consultant's reports. SPD evaluation of these materials and the issues follow.

FOUNDATION DISPLACEMENT CRITERIA.

2. In your letter, guidance was provided that we should not completely dismiss RCC until more specific detail on foundation conditions are available to verify

the potential for up to 4 feet in seismically generated foundation displacement as a design criteria, and that this approach would permit us to change course if the criteria could be relaxed. Our endorsement to the District of your letter provided a requirement for the District to prepare a Phase I assessment report, addressing the technical feasibility of constructing both dams based on the latest geologic and seismic conditions known for the site and to have this report reviewed by expert consultants. A Phase II Cost Comparison Report of both alternatives would be prepared following establishment of RCC feasibility.

3. During the Phase I evaluation it became obvious that the 4 foot displacement criteria was the overriding constraint in considering an RCC structure versus an earth-rockfill design. While two of the consultants questioned the need for such a conservative criteria, there is consensus among all consultants that an RCC design cannot be developed, even with mitigation, given this criteria. Conversely, the two structural consultants think an RCC design is feasible if the criteria were 1 foot or less displacement.

4. This prompts the question as to what information would be sufficient to persuade us to relax the criteria.

a. Dr. Clarence Allen has stated that "--if one could demonstrate that the local bedrock was completely unfractured, or that unbroken Pleistocene terraces covered the entire site area, this degree of conservatism might not be necessary, but such is not the case." It is also reported by George W. Housner that in his discussions with Dr. Allen, Allen allegedly stated that "in-depth geologic investigation of the site would likely provide evidence that would warrant reducing the four foot displacement but not below one foot."

b. Dr. Bruce Bolt indicates that "the detail of faulting in the exposed foundation rock after the excavation would be the crucial factor in further evaluation of this question."

5. Presumably, direct observation of in situ conditions along with a total geologic investigation program might allow us to consider a reduced displacement design criteria. However, there is no guarantee that this direct observation along with the results of in-depth geologic investigations would warrant a reduction of from four feet to one foot or less, even after in-depth evaluation with the related time and money impacts. Therefore, the likelihood of obtaining technical agreement on any significant reduction (to one foot or less) in the surficial foundation displacement criteria is remote. Without technical community agreement on such a large reduction, rational decision making requires that we adopt the more conservative criteria. Furthermore, it does not appear that extensive site investigation work aimed only at a small potential reduction in surficial displacement criteria is warranted during GDM development.

TECHNICAL FEASIBILITY.

6. The issue of technical feasibility of the two concepts seems more clear-cut. Both the RCC and earth-rockfill concepts presented in the Phase I Assessment - including suitable mitigation measures to attempt to meet the seismic design criteria of an 8+ Magnitude earthquake and up to four foot surficial foundation displacement, were reviewed by the Portland District,

consultants and this office. There is unanimous agreement that there are currently no mitigation measures that could assure the proper functioning of the 550' high RCC dam under these conditions. Making a major jump in scale from existing experience of approximately 300'+ high RCC dams to a 550' high RCC dam and doing it in a seismically complex geologic setting, in our view, pushes it beyond the "state-of-the-art".

7. There is unanimous agreement that the earth-rockfill embankment is technically feasible for this site with appropriate defensive design measures that are well accepted by the technical community. However, one consultant, Ray Clough expresses concern as to whether "the safety of an embankment dam could be assured during a four foot base displacement applied in an arbitrary direction while maintaining a full pool, and I expect the full pool requirement would be abandoned if such displacements must be accommodated."

8. This concern introduces the next issue, but does not change the conclusion that because of the overriding 4 foot foundation displacement criteria, the design concept that should be pursued during GDM development is the earth-rockfill embankment one.

RESERVOIR WATER SURFACE ELEVATION ASSUMPTIONS.

9. The District has established "water at or near spillway crest" criteria for purposes of dynamic analyses of both RCC and Earth-Rockfill alternatives during a seismic event. This criteria is a matter of long-standing CESPD/SPL policy established following the 1971 San *Fernando* earthquake and used in our Dam Safety Assurance Program (DSA). During our review the issue of risk assessment for the joint occurrences of independent events (earthquake and flood) was considered to try to arrive at the probability of such an event. Ray Clough has noted his concern (paragraph 7) on what he considers to be an overly conservative water surface elevation assumption criteria. He also notes that "the probability of an earthquake that produces fault displacement beneath the dam occurring during the very limited intervals of time when the reservoir would be full is so remote as to be completely negligible." The logic is that if it is negligible, then it need not be considered, and only lower pool levels need be of concern. With a lower pool concurrent with an earthquake, the RCC alternative becomes less complicated for identification of mitigation measures.

10. This issue was not addressed by the Los Angeles District, but has been evaluated by SPD staff. The level of risk (probability) for the joint occurrence of an 8+ Richter Magnitude (90% chance in 50 years) and a range of pool levels up to spillway crest (from 50-year recurrence interval flood to the SPF 333-year recurrence interval flood) were used and their joint occurrence probability was determined in accordance with the WES Letter Report "Joint Probability of Earthquake and Flood Events" dated 31 March 1978. These probabilities (see reference 18.d. attached) were calculated and plotted as a function of Reservoir Elevation corresponding to various flood events and flood durations (fractional weeks). From these data it is determined that the probability of a design earthquake occurring during a flood event that puts water at or near the spillway crest (elevation 2520) is approximately $1X10^{-7}$ in any year or $1X10^{-5}$ over the 100-year project life. This probability ranges up to about $8.5X10^{-4}$ for a flood event putting water to elevation 2360 (about 60% of height of dam).

CESPD-ED
SUBJECT: Roller Compacted Concrete (RCC) Alternative for Seven Oaks Dam, Santa Ana, California

11. The joint occurrence probability is not overly sensitive to water surface elevation/flood event and ranges between 10^{-5} to approximately 10^{-3} for the SPF (333-year event) to the 50-year event. Ray Clough has stated that these are rare events, and we agree; however, from a risk assessment point of view they may not be negligible. An acceptable level of risk has not been established by the technical community. Use of conservative design criteria that requires a safe structure at a joint occurrence probability of 10^{-5} may provide us with some degree of comfort, especially where it involves actions of a public body making decisions for others who have no direct control over the outcome of those decisions.

12. The true probability of an earthquake of magnitude 8+ occurring on the San Andreas fault with the epicenter in a location in close proximity to the site such that 4 foot surficial foundation displacement would take place at the site would probably be less than the predicted 90% in the next 50 years. This probability is for the epicenter to be anywhere along a 100 mile + fault length. Therefore the joint probability of a major flood event and a design earthquake producing 4 foot displacement would also be less. Conversely, the "repairability" of the structure ought to be considered, such that the time at which a flood event could be concurrent with an earthquake would be the time it would take to make major repairs to a structure following an 8+ magnitude event, which may be of several months duration. This would tend to make our joint probability risk assessment less conservative and would effectively counter the previous argument.

13. It is apparent to us that there is no specific way to quantify these factors, and therefore the conservative side of the currently available joint occurrence probability is considered appropriate and therefore stability analyses should be based on water being at the spillway crest.

COST COMPARISON.

14. In your guidance letter, you indicated "little doubt that a RCC gravity dam design would be less costly than the proposed scheme for an embankment dam." To respond to this concern we structured the evaluation in two phases, with the Phase II - Cost Comparison portion of their Assessment Report to be completed only if RCC is determined to be technically feasible. Because we have determined RCC to be not technically feasible at the Seven Oaks site for the criteria established, the Phase II Cost Comparison report has not been prepared.

15. We have not been able to define suitable mitigation measures for the RCC alternative that would assure a proper functioning dam under established criteria, and therefore costs which were earlier developed for this alternative do not represent a complete design concept. However, to put comparative costs in perspective the following data is forwarded. The cost data that are available consist of the Supplemental Phase I GDM cost estimate for the construction of an earth-rockfill dam which was $304 million. The Phase I GDM does not evaluate an RCC dam alternative. At the Phase II GDM General Design Conference in October 1986, a cost estimate of an RCC dam at the Seven Oaks site without mitigation was provided in the "Supplemental Information Paper, Roller Compacted Concrete Cost Estimate". The estimated cost of constructing an RCC dam was approximately $275 million. Subsequently added mitigation

measures that only partially meet the 4-foot seismic displacement criteria were costed. The cost of these partial mitigation measures raised the cost of the RCC dam to approximately $290 million, leaving an estimated difference of approximately $14 million less than the earth-rockfill embankment. If additional mitigation measures could be identified that would demonstrate technical feasibility, they would probably make the costs comparable, thus negating the primary reason for considering an alternative to the more conservative earth-rockfill embankment.

VIABILITY OF PARALLEL DESIGNS.

16. Your guidance was to not completely dismiss the RCC design until much more detail is known regarding specific foundation conditions. At this point it is the opinion of SPD that substantial additional exploration of the dam foundation would be required before any consideration could be given to easing the 4 foot displacement criteria. Information already gained from core drilling and geologic mapping at the site shows "evidence of many episodes of shear displacements along well defined as well as insignificant discontinuous surfaces". It is our opinion that additional geotechnical information will not give cause to a relaxation of the displacement criteria to 1 foot or less. Further, the cost and project schedule impacts of letting a large site investigation contract during GDM development (which is now under preparation) for work scheduled to be accomplished during Feature DM development is severe, and not considered prudent to undertake since the results are not expected to reduce the displacement criteria to one foot or less. An approach that would expose the entire foundation before considering a relaxation of criteria is also inappropriate because of differing water diversion schemes for the 2 concepts. A tunnel diversion could be used for the RCC design if a change were made following diversion and exposure of the foundation. However, it would add an approximate $30 million to the RCC cost and negate the cost advantage reason for the change. Therefore continuing parallel designs does not appear to have any cost or time advantages, and in fact may contribute to loss of GDM schedule efficiency and increased cost of the Seven Oaks Dam.

CONCLUSIONS

17. It is our view that the above issues are the key matters to be considered in determining the course of action on the RCC alternative. The references below provide substantial additional material and should be read in their original form to avoid misinterpretation and out of context quotations. CESPD considers the 4-foot displacement criteria and the "beyond the state-of-the-art" nature of a 550 foot high RCC dam to be the key factors which strongly suggest against an RCC design concept for the Seven Oaks site. We agree with the general consensus of all concerned that an RCC design concept cannot currently be developed that meets the specific criteria established for this site. Therefore we intend to proceed with design of an earth-rockfill concept. Your concurrence is requested. We are available to provide a briefing to your technical staff and discuss the issues in more detail. Arrangements for this briefing will be made by Mr. Frank Krhoun, (415) 556-5984. Suggest that it may be appropriate to invite Programs, Civil Works and Policy to the briefing for information purposes.

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SUBJECT: Roller Compacted Concrete (RCC) Altenrative for Seven Oaks Dam, Santa Ana, California

18. References:

a. Letter, DAEN-ECE-B, 10 October 1986, subject as above.

b. Endorsement, SPDED-PC dated, 26 November 1986, same subject.

c. Endorsement, SPLED-DA dated 10 April 1987, same subject, with attached Phase I Assessment, Embankment and Roller Compacted Concrete, Seven Oaks Dam, 31 December 1986.

d. Internal SPDED Memo, E. Morton Markowitz, 8 January 1987, subject: Seven Oaks Dam, Joint Probability of an Earthquake Striking a Reservoir Full of Water.

FOR THE COMMANDER:

A. E. Wanket

A. E. WANKET
Chief, Engineering Division

2 Enclosures
1. Ph I Rpt w/Appendix
2. Internal SPDED Memo, "Joint Prob."

CEEC-EB (CESPD-ED/9 Jul 87) (335-2-5c) 1st End
SUBJECT: Roller Compacted Concrete (RCC) Alternative for Seven Oaks Dam, Santa Ana, California

HQ, U.S. Army Corps of Engineers, Washington, D.C. 20314-1000 18 September 1987

FOR: Commander, South Pacific Division, ATTN: CESPD-ED

1. I concur with your intention to proceed with the earth and rockfill concept. However, if significant reductions in the established design criteria (four-foot foundation displacement in any direction resulting from an 8+ Richter Magnitude seismic event with the reservoir surface at the spillway crest) result from further explorations, investigations, or studies during the development of the dam, the roller compacted concrete alternative may again become technically feasible and deserving of further consideration.

2. The briefing on this subject by Mr. Walter Day on 17 September 1987, was comprehensive, timely and well received. Both Mr. Day and other division and district personnel involved in the study should be commended for an excellent effort on a tight schedule.

FOR THE COMMANDER:

2 Encls
wd all encls

WILLIAM N. McCORMICK, JR.
Chief, Engineering Division
Directorate of Engineering and Construction

CESPD-ED-G (CEEC-ED/9 Jul 87) (1130-2-305b)2nd End Parrillo/am/6-5
SUBJECT: Roller Compacted Concrete (RCC) Alternative for Seven Oaks Dam, Santa Ana, California

DA, South Pacific Division, Corps of Engineers, 630 Sansome Street, Room 720, San Francisco, CA 94111-2206 2 ... 1987

FOR: Commander, Los Angeles District, ATTN: CESPL-ED

1. For your information and implementation.

2. The District should proceed with development of the GDM based on an earth and rockfill structure. However, pursuant to para. 1 of 1st End, a review of this decision should be scheduled when results of additional investigations are available. A milestone event should be developed to address this review at latest after completion of explorations for the Embankment Feature Design Memorandum.

3. Conclusions on the RCC Alternative should be presented in the Phase II GDM. This complete package, including enclosures to all endorsements, District's initial evaluation and technical consultants' reports should be made an appendix to the Phase II GDM.

FOR THE COMMANDER:

A. E. WANKET
Chief, Engineering Division

PHASE I ASSESSMENT

EMBANKMENT AND ROLLER COMPACTED CONCRETE DAMS

SEVEN OAKS DAM

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ATTACHMENTS

1. DEN-ECE-B, dated 10 October 1986
2. SPDED-PC (DAEN-ECE-B) 1st End, dated 26 November 1986
3. Clarence R. Allen, Letter dated 8 December 1984
4. Portland District Views, NPPEN-GE, letter dated 10 December 1986, Subject, Santa Ana Project, Seven Oaks Dam, Concrete Gavity Dam Evaluation.
5. Consultant view H. Seed, letter dated 19 February 1987
6. Consultant view R. Peck, letter dated 5 February 1987
7. Consultant view G. Housner, letter dated 22 March 1987
8. Consultant view R. Clough, letter dated 30 March 1987
9. Consultant view B. Bolt, report dated 30 March 1987

Executive Summary

The Phase I assessment indicates that either an embankment dam or a roller compacted concrete (RCC) dam could be designed and constructed at the site. However, an RCC dam could be constructed at the Seven Oaks damsite only with an array of defensive measures added to the RCC dam which would mitigate, only in part, the effect of postulated foundation displacements. A feasible defensive measure was not developed for random cracking within the RCC dam. Well established and precidented defensive measures have been included in the earth-rockfill embankment design. Literature review indicated that concrete dams should not be considered at sites where displacement in the foundation is foreseen.

Previous cost studies, without defensive design measures for the RCC dam, show that the cost estimates are reasonably comparable for the two types of dams, and the costs are anticipatd to remain comparable with the inclusion of the defen ive measures for the RCC dam.

The high probability of a great earthquake in the immediate proximity of the dam resulting in a significant postulated displacement in any direction leads this District to the conclusion that a properly designed and precidented earth rockfill embankment dam would be preferable. The flexibility of an embankment dam, with the ability to heal itself when damaged, is highly desirable for this location. Conversely, the rigidity of the RCC dam could lead to cracking due to postulated differential movement in the foundation, at the abutments, and within the concrete structure. It is the conclusion of the Los Angeles District that an earth-rockfill embankment dam is the prudent and appropriate type for the Seven Oaks damsite.

SEVEN OAKS DAM

PHASE I ASSESSMENT

1.0 PURPOSE. The purpose of this paper is to provide an assessment of an earth-rockfill embankment dam and a roller compacted concrete (RCC) dam at the Seven Oaks damsite. The assessment is required since a previous evaluation and cost estimate indicated a potential cost savings for the RCC alternative. The previous cost estimate for the RCC dam did not include defensive measures for effects of postulated displacements. This study addresses technical considerations only.

2.0 BACKGROUND. Reference is made to the Supplement to the Phase I GDM on the Santa Ana River Main Stem dated December 1985, in which the Los Angeles District proposed an earth-rockfill embankment dam as the upstream element for flood control storage. During the review process of the Supplement, a comment by the structural elements of OCE requested an evaluation of an RCC alternative in that an RCC gravity dam may be less costly than the proposed embankment dam. An information paper titled "Roller Compacted Concrete Vs Embankment Comparative Study" and a "Cost Estimate" were prepared for the Seven Oaks Dam General Design Conference of October 1986. Subsequent to the October conference, OCE and SPD requested an assessment be prepared by the District (attachments 1 and 2). This paper is the first phase of that assessment. The guidance requires the views of the Portland District and independent technical experts. The views of the Portland District are included as attachment 4, those of the technical experts are included as attachments 5 through 9. The views of the experts are based on design concepts for RCC presented in this paper, which were not fully developed at

the time of the review by the Portland District. The Portland District comments are based on review of the supplement to the Phase I GDM, the comparative study and the cost estimates.

3.0 DESIGN CRITERIA.

3.1 Embankment Dam. The embankment design slope stability criteria is set forth in:

(1) EM 1110-2-1902, which requires stability evaluation for the following conditions:

End of construction,

Steady seepage with surcharge and maximum pools,

Sudden drawdown from spillway crest and maximum pools, and

Seismic loading at end of construction, normal (partial) and maximum storage pools, and

(2) ER 1110-2-1806, which provides general guidance for the seismic evaluation of both embankment and foundation.

In addition to the above, when conducting a dynamic analysis, in areas of high seismic activity, the SPD/SPL policy is to evaluate the design for the concurrent occurrence of the design earthquake event with the pool at or near spillway crest. This policy was adopted subsequent to the San Fernando earthquake of 1971. It was the criteria used for all the District's dams analyzed under the Dam Safety Assurance Program (DSAP). This policy will be continued to be used in the assessment of both dams. It should be noted there are no criteria available dealing with potential foundation displacement.

3.2 Concrete Dam. The earthquake analysis and design of concrete gravity dam criteria is set forth in ETL 1110-2-303 which provides the following guidance:

3.2.1 Stability Analysis. The seismic coefficient method of analysis will continue to be used to determine overturning and sliding stability with the following exceptions: (1) the seismic coefficient used in the analysis should be no less than that given in ER 1110-2-1806, and (2) the sliding stability analysis should follow ETL 1110-2-256. The ETL 1110-2-256 requires a minimum factor of safety of 2.0 for normal static loading and a minimum factor of safety of 1.3 for seismic loading.

3.2.2 Internal Stress Analysis. Internal stresses shall be determined using a dynamic stress analysis.

3.2.3 Structural Analysis. The structural analysis for earthquake loadings consists of two parts: (1) the traditional overturning and sliding analysis using an appropriate seismic coefficient, and (2) a dynamic internal stress analysis using site dependent earthquake ground motions. In both cases the selected pool level should be the one judged likely to exist coincident with the selected design earthquake event. This means that the pool level selected for an earthquake loading case by criteria should normally be a pool level which occurs, on the average, relatively frequently during the course of the year. However, as stated in paragraph 3.1, the SPD/SPL policy is to evaluate the design for the concurrence of the design earthquake event with the pool at or near spillway crest. This policy will continue to be used in the assessment of both dams.

4.0 REGIONAL GEOLOGY. Upper Santa Ana River Canyon drains the southerly facing slopes of the San Bernardino Mountains. This mountain block represents the eastern segment of the Transverse Ranges physiographic and tectonic province. The geomorphic and structural features in this province lie across the grain of adjacent provinces, which are strongly influenced by the northwest trending San Andreas fault system. The principal rock types found within the San Bernardino Mountains are Precambrian plutonics and metamorphics and complex sections of Cretaceous and younger rocks. The southerly margin of the mountains is distinctly separated topographically from the broad, alluvial fans at the base by the South Branch of the San Andreas fault.

5.0 REGIONAL TECTONIC SETTING. As opposed to other areas in California along the San Andreas fault, where the fault's motion can be simply defined by right lateral motion along a major fault trace, the tectonic history of the San Andreas system in the vicinity of the damsite is complex. Although it is widely recognized that the South Branch, at the mouth of Santa Ana River Canyon, is the neotectonic trace of the San Andreas fault, the most recent researchers (Matti, Morton and Cox, 1985) believe that other strands of the fault have had much more activity and have contributed many more miles of right lateral displacement over the last 4 million years. These strands, identified as the Mill Creek, Wilson Creek and Mission Creek individually and sometimes collectively at the North Branch, are apparently inactive and are located to the north-northeast of the presently active trace. It is believed that the complexity of the faulting and the movement of fault activity to the southwest are due at least in part to the influence of the left lateral Pinto Mountain fault, which has acted to move the San Bernardino Mountains across the trend of the San Andreas system.

One result of this shift of fault activity has been to juxtapose a fault wedge of exotic plutonic and metamorphic rocks, the Wilson Creek block, against the southerly flank of the San Bernardino Mountains. It is within this wedge of far traveled rock that the damsite is located. Because the footprint for the RCC dam falls within the larger footprint of the embankment dam, both damsites will be referred to singularly as the Seven Oaks damsite.

In the area of the damsite, the active South Branch is also known as the San Bernardino segment. Although the fault's movement is still primarily right lateral, a vertical component of movement is suggested by the high angle northeast dip of the fault at the mouth of Santa Ana Canyon and the abrupt relief of the San Bernardino Mountain front.

6.0 SITE GEOLOGY. The damsite is located approximately one mile upstream from the mouth of Santa Ana River Canyon and the South Branch of the San Andreas fault. The North Branch of the fault crosses the reservoir area about 1/2 mile upstream from the proposed damsite. The wedge of rock between the two branches of the fault in the vicinity of the damsite is composed of Precambrian (?) gneiss and medium grained Cretaceous diorite (Kqd). Portions of the gneissic rockmass display a strong northeasterly to easterly foliation which dips steeply to the south. The gneiss is often highly sheared, jointed and fractured. Prominent near-vertical planar discontinuities trend west-to-northwest. The diorite rockmass is likewise strongly jointed and fractured. It is also slightly foliated. Isolated inclusions or roof pendants of gneiss are exposed within the diorite. Exploratory coring has verified the fractured nature of the rockmasses to depths of several hundred feet. Many discontinuities show signs of movement as evidenced by thin clay gouge and slickensides. For example, core from the most recent exploratory

hole at the left abutment (C86-4) is considered to be generally representative of the foundation rockmass at the damsite. The hole was drilled to a depth of 400.5' and 214.0' of HQ core was recovered. Core recovery percentage was 79 percent and the cumulative RQD was 18 percent. Out of 1038 discontinuities counted (including recemented fractures and joints), 65 had slickensides. The orientations of the sheared discontinui-ties varied from 0^{o} to 90^{o} with most between 40^{o} and 65^{o} from horizontal. In addition, eleven gouge zones with orientations of 15^{o} to 80^{o} were encountered.

7.0 SITE FOUNDATION CONDITIONS. At the damsite, the Santa Ana River has eroded an assymetrically shaped canyon into the highly fractured and surficially weathered diorite and gneissic bedrock. In the canyon, coarse flood deposited alluvium has accumulated to depths in excess of 100 feet. At various elevations on the canyon walls, remnants of coarse alluvial terraces are locally preserved.

The abutments at the site are primarily diorite. The rockmass at the right abutment is moderately well exposed, except for terrace remnants at the higher elevations. The left abutment, on the other hand, is covered by colluvium and the diorite bedrock is exposed only at the higher elevations and along a couple of rock ridges. It has been estimated that as much as 20 feet of weathered rock at both abutments and at the left abutment as much as 110 feet of colluvium of the embankment damsite will require stripping to expose a suitable dam foundation. The RCC site which is several hundred feet downstream from the embankment centerline has colluvial cover. The estimated average thickness is 20 feet as opposed to an average of 40 feet at the embankment site.

The depth of streambed alluvium, which would require removal at the site varies generally between 20 and 100 feet. Five to twenty feet of bedrock (primarily diorite) may need removal prior to surface foundation treatment.

8.0 SEISMICITY. Because the damsite is located in the area of the "big bend" (between the Garlock and Pinto Mountain faults), where the San Andreas fault appears to be "locked", the instrumentally recorded seismicity in the vicinity of the damsite is relatively low when compared with other areas in southern California. Between 1932 and 1985, the closest event of magnitude 5-or-greater (October 1985, M 5.0) was centered over 11 miles from the site. The great 1857 Fort Tejon earthquake was the last major earthquake along the San Andreas to affect the damsite. The mean recurrence interval for these great earthquakes appears to be on the order of 150 to 200 years. Professor Kerry Sieh rates this particular segment of the San Andreas fault as the most likely to break in association with a great earthquake in the near future, with a probability of 50 to 90 percent within the next 50 years. Undoubtedly, the site will be subjected to shaking on a more frequent basis due to small-to-medium-magnitude earthquakes on more distant faults; however, even the attenuated accelerations from these events will be insignificant by comparison to a near-field great earthquake on the San Andreas fault.

9.0 DESIGN EARTHQUAKE. It is conceded in the geotechnical community that the San Andreas fault in southern California is capable of generating a magnitude 8+ earthquake. Though very little information exists on the expected accelerations produced by a great earthquake in the near-field, it can be assumed for design purposes that the peak bedrock accelerations at the damsite produced by a magnitude 8+ earthquake with an epicenter at the mouth of the canyon may exceed 0.8g, and the duration of strong shaking in excess of 0.05g will be approximately 60 seconds.

10.0 SITE DISPLACEMENTS. Dr Clarence Allen of the California Institute of Technology, in his 1984 report on the damsite, states: "An added complication for the new site is that, because the San Andreas fault dips about 65 degrees north at the mouth of the canyon, the site may be on the upthrow block of a thrust fault and is therefore, in a somewhat more vulnerable location from the point of view of subsidiary faulting than are locations on the downstream block to the south (Sherard et al. 1974)." The reference goes on to state that "Areas surrounding such sites can be expected to be distorted during earthquakes originating on the main faults, and it is impossible to be confident that small displacements could not occur on almost any existing rock shear zone or fracture, or even possibly new breaks." Based upon the site geology and potential seismic activity, Dr. Allen stated: "..I recommend designing the "Seven Oaks Dam" on the assumption that as much as 4 feet of surficial fault displacement in any direction could take place arbitrarily beneath the facility...". See Attachment 3.

Geologic mapping and exploratory coring conducted at the damsite supports this assumption. The rockmasses at the site, and probably throughout the Wilson Creek block, show evidence of many episodes of shear displacements along well defined as well as insignificant discontinuous surfaces. While it is reasonable to expect that movement may occur on one of innumerable planes throughout the damsite vicinity during a major earthquake near the site, it could likewise be conceived that displacements will vary from insignificant to possibly as much as 4 feet along a number of shear planes.

11.0 PRELIMINARY DESIGN CONSIDERATIONS

11.1 **Description Embankment Dam.** The recommended Seven Oaks Dam is an earth and rockfill embankment with a maximum height of 550 feet above streambed located at streambed elevation 2060 feet on the upper Santa Ana River (see plate 1). The dam has a crest elevation of 2610 feet, a crest width of 40 feet and a crest length of about 3000 feet. Typical embankment cross sections are presented on plate 2. A detached, unlined spillway with a crest elevation of 2580 feet and a crest width of 500 feet would be located about 1700 feet east of the dam in a natural saddle. The outlet would be located in the left abutment with an intake elevation of 2100 feet. This tunnel, 15 feet in diameter and approximately 1900 feet in length, would be excavated in rock and lined with reinforced concrete. Both types of dams would provide a storage of approximately 160,000 acre-feet.

Design parameters and calculations for the embankment dam and the appurtenant structures are provided in the referenced Phase I GDM Supplement. Additional data are being developed for the Phase II GDM.

11.2 **Description RCC Dam.** The RCC dam is located at the same site as the embankment dam, see plate 1 for relative location of the centerline. The RCC gravity dam would have a maximum height above the bedrock foundation of about 600 feet and above the streambed of about 500 feet. The dam has a crest elevation of 2550 feet, a crest width of 20 feet, an arched crest length of about 2100 feet, and a spillway crest at elevation 2520. The cross section would have a vertical upstream face and a downstream slope of 1 vertical on 0.9 horizontal, see plates 3 and 5. This slope is required for sliding and structural considerations.

Internal features of the dam, see plate 5, consist of a formed upstream facing of conventional concrete 3-foot thick backed by a 9-foot thick layer of special RCC to assure a water tight face and a formed spillway facing concrete 3-foot thick. The final designed zoning would be based on internal stresses determined by dynamic analysis. Foundation concrete with an average 5-foot thickness was assumed for leveling of the streambed foundation and 3-foot thickness for protection of the abutments. No special facing or forms would be used in the downstream non-overflow sections of the dam. Conventional concrete would be placed around the gallery and outlet works. Transverse vertical contraction joints would be placed at about 50-foot spacing across the dam. An RCC stilling basin 350 feet long and 500-foot wide is provided at the toe of the dam. The RCC invert slab would be 20-foot thick for 200 feet from the face of the dam and 10-foot thick for 150 feet. Stilling basin walls would be gravity RCC construction about 60 feet high.

12.0 STATIC STABILITY.

12.1 Embankment Dam. In the Supplement to the Phase I GDM, the proposed embankment was analyzed for end of construction, steady seepage, rapid drawdown and seismic loading conditions in accordance with the requirements of EM 1110-2-1902, "Engineering and Design, Stability of Earth and Rockfill Dams." The analyses were performed with the Corps of Engineers computer program UTEXAS2, using Spencer's Method and appropriate assumed material properties. A maximum pool elevation of 2500 feet, corresponding to about a 200-year flood, was used for steady seepage and rapid drawdown conditions since higher pool elevations will not have durations sufficient to establish saturation of the embankment. Calculated safety factors for each condition are higher than the minimum required by the criteria for both the upstream and downstream slopes.

12.2 Roller Compacted Concrete Dam. The RCC gravity dam was analyzed for a height of 600 feet above the bedrock foundation with a base width of 540 feet, a 20-foot crest width and a 1 vertical to 0.9 horizontal downstream slope and foundation drains located 30 feet from the upstream face. The static stability for sliding and overturning of the dam using foundation contact parameters of friction equal to 45° and cohesion equal to 100 pounds per square inch is adequate for all loading conditions. The maximum foundation pressure at the upstream toe for various design loadings would be about 115,000 lb/ft^2 and at the downstream toe would be about 45,000 lb/ft^2. Although the rockmass at the foundation is severely fractured and jointed based on unconfined compressive strengths of the bedrock cores, bearing capacity appears to be adequate. For seismic stability, a pseudo-static method of analysis assuming an intermediate water level at elevation of 2300 feet (about a 30 year flood event) or about 250 feet from the top of dam and using a pseudo-acceleration of 0.2g would have an adequate sliding safety factor of about 1.9. With the pool at spillway crest a peudo-acceleration of 0.2g would have a sliding safety factor of about 1.4. Both cases were analyzed using a drained foundation case.

12.3 DISPLACEMENT. It has been recommended that the Seven Oaks Dam be designed with the assumption that as much as 4 feet of surficial fault displacement in any direction could take place arbitrarily beneath the facility. Displacements of this magnitude are expected to cause damage to the foundation and structure which can not be prevented. Various "worst case" depictions of foundation displacements and structure response are shown on plate 6.

Although the predominant movements during a major earthquake along the San Andreas fault would be lateral (strike-slip), components of vertical uplift (reverse or thrust) are also possible. In either event, the rockmass at the damsite will be subjected to movements of various orientations and magnitudes as it compensates for the expected large movements along the main fault rupture surface. Normal, reverse, and lateral movements must all be considered possible.

A rigid structure such as an RCC dam subject to fault displacements of these types could be subjected to tremendous strains caused by the relative displacements of the abutments (see plate 6). In the case of normal fault, an increase in the horizontal distance between the two abutments may occur. This could result in opening of construction joints, cracks, or separation of the dam from the abutment foundation. In the event the dam "bridged" the abrupt displacement, the dam could be separated from the foundation and a conduit could exist from the upstream to the downstream toe of the dam rendering the foundation drains ineffective and causing erosion of the foundation. These movements could result in serious damage if a retained pool exists behind the dam and erodes the foundation weakened by the fault. In the case of reverse fault movements, compression forces may be created in the dam which could lead to crushing and cracking of the dam. Lateral fault movements also could lead to crushing and cracking of the foundation and concrete within the dam. In addition, lateral offsets could lead to an increase in the horizontal distance between the two abutments.

In almost all cases, severe damage is inevitable if a structure is intersected by a fault break, in that even small displacements can generate very high stresses in rigid structures.

The following quotes are from the referenced paper by Sherard, Cluff and Allen:

"Where demonstrably active faults cross the foundation, concrete dams should not be considered. This is the strong opinion of the Authors and is believed to be the general opinion in the industry. Embankment dams can be designed with confidence to withstand safely the deformations (and damaging results thereof) imposed by fault movement, whereas any design of a concrete dam on an active fault is questionable."

While no active fault exists in the foundation of the proposed dam, a 4-foot displacement in any direction representing sympathetic movement is postulated for this site.

"For a concrete gravity dam, fault movement can break the contact between the dam and the foundation, allowing full uplift pressure to act underneath, reducing the shearing resistance along the dam base and causing failure by sliding on the foundation of the general type which resulted in the disaster at the St. Francis Dam in Southern California in 1928. For a concrete arch dam an abrupt displacement of two parts of the dam of the order of 0.25-0.50 m in almost any direction could cause complete sudden failure by several mechanisms. The concrete could be crushed or one end of the dam lifted off its abutment, destroying the continuity of the arch", and "The Authors believe that concrete dams should also generally be avoided in earthquake regions at sites with faults for which the studies of the activity are inconclusive."

Protective measures would be required to minimize the effects of these displacements to the concrete dam. The impact of the displacements would be most severe if they occur concurrent with the pool at or near spillway crest, or about 570 feet of static head at the foundation level.

13.0 ASSESSMENT OF DYNAMIC RESPONSE

13.1 Embankment Dam. The response of the proposed earth-rockfill embankment to strong shaking at the damsite was addressed specifically for this assessment using simplified seismic evaluation procedures.

13.1.1 Crest Accelerations. The accelerations of the embankment during the San Andreas event (maximum credible event) were determined using the one-dimensional wave propagation computer program, SHAKE. Using SHAKE, it was possible to select a representative earthquake motion at the bedrock surface and propagate the acceleration response through the overlying soil layers.

The hypothetical accelerogram developed by Seed and Idriss (1969) known as the Seed-Idriss record was scaled to 0.8g peak horizontal acceleration and propagated from the bedrock surface through soil profiles of the embankment crest and pervious shell mid-slope. The results of the embankment response analysis indicate that accelerations attenuate (decrease) significantly at the crest and mid-slope. The reduction in accelerations indicates that much of the energy imparted by the earthquake shaking has been damped (absorbed) by the soil layers.

13.1.2 Strains Caused by Displacements. The embankment dam would, for the most part, behave as an elastic structure and would develop varying stresses and strains from the earthquake shaking. Calculating the magnitude of these stresses and strains is beyond the scope of this study but will be determined by finite element procedures later in the FDM studies. Should induced stresses and strains develop in the embankment to a state of "instability", the embankment materials would deform to release these strains and establish a condition of stability.

13.1.3 Potential for Cracking. The embankment will settle, and will continue to settle at a decreasing rate for many years. The amount and rate of settlement will depend on the compactive effort, lift thicknesses, moisture content at placement, and in the case of the quarried shell materials, the amount of rock breakage. The potential for settlement will be evaluated during design and will be minimized by varying and controlling the above factors.

13.1.4 Settlement will occur, however, cracking in the embankment will be minimized by defensive design measures as described in paragraph 14.1.

13.2 RCC Dam. Calculated stresses within gravity dams using standard design loads have little resemblance to those predicted by a dynamic response analysis of the dam to earthquake motion. Predictions of the magnitude of these strains and stresses within the dam would require a detailed dynamic analysis which is beyond the scope of this study. The design strength properties of the concrete would be established from the results of the analysis. In order to assess maximum principal stress development in the dam, a simple single degree of freedom (SDOF) analysis was conducted using Seed's mean design spectrum for rock sites with a peak ground acceleration of 0.8g, and a pool elevation of 2520 feet (spillway crest). The analysis provides the maximum principal stresses on the upstream and downstream surfaces of the dam. A maximum principal tensile stress of about 1200 psi was calculated on the upstream face at the foundation level and 1250 psi at 375 feet above the bedrock foundation on the downstream face (f^1_c = 4000 psi). With the pool at spillway crest and using a peak ground acceleration of 0.5g, a maximum principal tensile stress of about 700 psi was calculated on the upstream face

at the foundation level and 650 psi and at 375 feet above the foundation on the downstream face. Based on these SDOF results, it appears the dam could be designed to elastically resist relatively moderate intensity earthquakes and designed for limited cracking for the maximum design earthquake.

14.0 DEFENSIVE MEASURES FOR EARTHQUAKE.

14.1 Embankment Dam. Given the extreme seismic environment and the relative abundance of embankment-type material at or near the site, a zoned earth and rockfill embankment is the preferred and prudent alternative due to the ability of a properly zoned embankment dam to seal cracks which might develop from strong shaking and ground movement in an earthquake. Embankment zoning and gradation requirements provide resistance to concentrated leaks by placing high strength and erosion resistant materials at the outer shells where they are most effective. The proposed embankment for the Seven Oaks Dam features crack-resistant impervious core and well-graded, cohesionless filter and transition zones effective in sealing internal cracks which may develop due to differential settlement, displacement, or earthquake induced ground movement. Sherard, Cluff and Allen 1974 state that: "Sands, gravels, sand-gravel mixtures and masses of hard rock fragments, when these materials are free of any appreciable admixtures of fine-grained soil (clays or silts), are cohesionless in the sense that each individual particle is not held to its neighbours. There is no cementation of any kind. Hence, a mass of these materials will not stand on an unsupported vertical face of any appreciable height and it is impossible for an open crack to exist in the mass. Any open crack which might tend to be formed momentarily by the fault displacement must immediately close by the action of the walls of the crack collapsing. Hence,

internal zones of these materials act as 'crack stoppers' in dam embankments." The cohesionless sands and gravels available for construction of the filter and transition zones of Seven Oaks Dam would be unable to sustain an open crack and would thus act to seal potential cracks which might develop in the core. Sherard, Cluff and Allen also state in their paper that: "With the cohesionless transition zones acting as crack stoppers, even if a large open crack develops in the impervious core the maximum leakage will be limited by the permeability of the transition material to a volume which can be controlled safely. The leakage is controlled by providing a zone of considerably coarser and more pervious material (usually quarried rock fill or screened large cobbles) downstream from the downstream transition zone, so that the leakage must pass from the transition zone into the rockfill zone, and exit from the dam through the rockfill zone. Since the permeability of the transition zone will always be at least an order of magnitude (and usually several orders of magnitude) less than that of the rockfill zone, the amount of leakage water will always fall well below the quantity which would tax the hydraulic capacity of the rockfill zone. Also, the relative gradation of the transition zone and the rockfill zones are made such that the individual particles of transition zone materials cannot penetrate the voids of the rockfill zone. Using these design principles, the maximum possible leakage quantity will emerge safely under complete control through the toe of the downstream rockfill zone." Dr. H. Bolton Seed of the University of California, Berkeley stated in his 1979 Rankine lecture: "...many of the potentially harmful effects of earthquakes on earth and rockfill dams can be eliminated by adopting defensive measures which render the effects non-harmful." Some of the defensive design measures which would be incorporated in the embankment are:

- using a zoned embankment section which includes a wide, elastically behaving central core with well-compacted, free-draining upstream and downstream shells and transition and filter zones to preclude piping due to cracking,

- providing a well prepared foundation-embankment contact to reduce seepage,

- increasing the core width and or the transition width at the abutment contacts,

- selecting adequate freeboard to prevent overtopping due to crest settlement, uplift of reservoir bottom, or waves generated by seiches and landslides, and

- using a wide and suitably armored crest to protect against very unlikely overtopping.

Dr. Seed also stated that: "Defensive measures, especially the use of wide filters and transition zones, provide a major contribution to earthquake-resistant design and should be the first consideration by the prudent engineer in arriving at a solution to problems posed by the possibility of earthquake effects." Sherard, Cluff and Allen made the statement that: "we are confident that conservatively designed embankment dams will withstand without failure the worst conceivable damage imposed by foundation fault movements of these expected magnitudes, provided that there are materials available from which to construct thick, protective zones of cohesionless transition material."

The types of material designated and the zoning of the embankment as presently proposed for the Seven Oaks Dam, are state-of-the-practice and conventionally used in the present day design of embankment dams to address displacements and for dynamic loadings in general. Therefore, those preventive measures are included and reflected in the embankment cost estimate presented in the supplement to the Phase I GDM.

14.2 RCC Dam. The dam can be designed for static, as well as dynamic conditions. It is anticipated that high internal principal tensile stresses will occur which can be accommodated with concrete zoning and modification of the dam cross section. "With regard to movements of faults in dam foundation, there is essentially no experience on record to guide us, so that related performance and judgment must be relied on" Sherard, et al 1974. Design considerations to minimize the effects of displacement are limited for concrete structures. A special joint has been used for concrete gravity dams where the location of the suspect fault was known (Morris Dam, California) There has not been any experienced movement associated with that joint since 1935 to evaluate its performance and effectiveness. Auburn dam in California was proposed with a special joint, but the dam has not been constructed. Once again the location of the suspicious fault was known.

To address displacement at the Seven Oaks site the following additional design measures, not yet included in the preliminary cost estimate, are recommended. These defensive measures are largely judgmental and untried, as there is no experience on record to provide guidance. The measures are intended to address the various displacements that need to be considered as occurring at any location within the foundation.

14.2.1 General. Given the height of the dam above bedrock to length of the crest ratio, arching of the axis was introduced to (1) improve the conditions at the abutments, (2) increase the downstream sliding resistance, and (3) put the vertical joints in compression. To provide additional sliding resistance for both the static and dynamic stability, the upstream and downstream foundation trenches excavated into bedrock would be backfilled with RCC concrete.

14.2.2 Foundation to elevation 2400. The following measures would be incorporated to address the worst case scenario of 4-foot displacement in the foundation. This differential movement could cause separation at the contact of the RCC with the foundation rock and openings along vertical joints or cracks in the concrete. A zoned buttress or "crack stopper" fill would be constructed in the streambed along the upstream face of the dam from bedrock to an elevation about 2200 (approximately a 10 year filling frequency). The fill would limit the flow of the reservoir through or under the dam in the event of displacements. The zoned fill would have a crest width of 15 feet, an outer slope of 1 vertical on 2.5 horizontal. The zoned fill would be extended up the abutments with a 1 vertical to 2 horizontal top slope to about elevation 2400. The cross section will have core, filter, transition, pervious and cobble size material, in that order, with the coarser material at the contact with the dam (see plate 5). The zoned fill is considered the most effective measure to deal with the foundation displacements and openings and is incorporated with diminishing height to approximately elevation 2400. The final selected height and configuration of the zoned fill would be based on consideration of its effect on the downstream factor of safety for sliding and seepage studies.

14.2.3 Abutments. To address separation and displacements at the abutments, the abutment treatment above elevation 2400 would consist of backfilling the upstream foundation trench with cobble size material. The cobble size material would act as a "crack stopper" in the event of abutment displacements and limit the flow of the reservoir through or under the dam. A grouted stone gutter would be placed over the trench backfill. The foundation backfill concrete (paragraph 14.2.1) would not be placed in this area. The gutter would provide erosion control at the contact and could be removed in the event of damage due to an earthquake to inspect the dam or foundation and perform any necessary repairs.

14.2.4 Vertical joints. Transverse vertical joints are provided at 50-foot spacing across the dam axis. It should be noted that joint spacing required for thermal contraction for RCC construction has generally been 200 to 300 feet in dams constructed to date in the United States. The joint spacing provided for the Seven Oaks Dam is intended to accommodate sympathetic displacements traversing the dam axis and to limit the extent of cracking that would develop within the dam. The vertical joints, in addition to having conventional waterstops at the upstream face, would have specially manufactured waterstops which would be designed to accommodate about four feet of movement in any direction. The special joint material would be a heavy ply elastic material with an unembedded length of about 1.5 feet and capable of supporting the full reservoir pool (see detail on plate 4).

The potential would still exist for cracking to occur in the concrete between joints. No feasible measure is known which can be constructed to accommodate random cracking occurring on the face of the dam. The occurrence of a random separation crack between joints with a full reservoir pool could result in serious damage to the foundation and structure.

14.2.5 Seismic Design Criteria. Due to the potential that the foundation drains would be rendered ineffective by displacement coincident with a full reservoir, it is recommended that the dam be designed with a safety factor against sliding of 1.3 including 100 percent uplift force. A reduction in the foundation strength properties would be required to account for the potential separation of the dam and foundation. The effect of these design criteria on the stability of the dam has not been evaluated.

15.0 SUMMARY. The previous comparative cost comparison of a roller compacted concrete dam and an earth-rockfill dam showed a potential savings of less than 10 percent for an RCC dam. The estimate for the RCC dam did not include the defensive measures described above. It is estimated that the inclusion of these additional measures would result in the cost of the two dams to remain comparable.

The difference between the two dams would be the degree of assurance of satisfactory performance during a major seismic event with a high reservoir pool. The vulnerability to damage also does not appear comparable for both types of dams. Accelerations from a very large magnitude earthquake on the San Andreas fault may exceed 0.8g at the damsite, and fault displacement at the damsite has been postulated to be 4 feet in any direction. The flexibility of an embankment dam, with the capability to heal itself when damaged, is highly desirable for this location. Separation of the RCC dam at the foundation and contacts is possible. Defensive measures are required for the RCC dam. The effectiveness of the defensive measures against foundation movements are untried and largely judgemental. A zoned buttress fill is considered the most effective measure to deal with the foundation

displacements and openings at the dam foundation. The zoned buttress fill would act as a "crack stopper" and control the release of reservoir through the foundation and random cracks at its contact with the structure. It is infeasible to install the zoned buttress fill for protection of the face of the dam to the full height and it is provided only to elevation 2200 in the streambed and with diminishing height on the abutments to elevation 2400. Defensive measures above the top of the buttress fill consist of backfilling the upstream foundation trench at the abutments with a cobble size material and providing 50 foot spacing of contractions joints with a specially manufactured waterstop within the dam. The effective functioning of the special waterstop is questionable, as it does not defend against all modes of movement or locations of cracking. There is no feasible measure known which can be incorporated in the design for the occurrence of random cracking within the structure. Hence, there is a lack of confidence in the performance of the RCC dam with a full reservoir and random cracking of the structure. The concrete gravity dam would also be designed for full uplift pressures and with a reduction of foundation strength parameters.

Based on consideration of the postulated displacements, the unconventional defensive measures required for the RCC design, the lack of effective defensive measures for random cracking within the RCC dam and the difference in behavior of the two types of dams an embankment dam is recommended for the Seven Oaks Dam.

ATTACHMENTS

NOV.26 '86 16:16 USAEDSP SPDHS SF CA

DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
WASHINGTON, D.C. 20314-1000

REPLY TO
ATTENTION OF:

DAEN-ECE-B

10 Oct 86
~~26 September~~ 1986

SUBJECT: Santa Ana River, CA

Commander, South Pacific Division
ATTN: SPDED

1. At the meeting of 10 September 1986 in this office on the subject project, comment was offered regarding consideration of a roller compacted concrete (RCC) gravity design concept for the proposed Seven Oaks Dam. The purpose of this letter is to describe the degree to which I believe this design concept should be pursued as work on the General Design Memorandum (GDM) proceeds.

2. There is little doubt that a RCC gravity dam design would be less costly than the proposed scheme for an embankment dam. However, due to seismic/foundation considerations that are known, as well as those which are strongly suspected but remain to be investigated in FY 1987 and FY 1988, we believe it prudent to proceed with the GDM on the basis of an earth-rockfill embankment dam, a design concept that can accommodate deformations that may approach the four feet that is recommended by Los Angeles District's seismic consultant. However, a RCC design should not be completely dismissed until much more detail is known regarding specific foundation conditions at and near the site. We believe that Los Angeles District should roughly scope a RCC design, and become more acquainted with RCC design and construction techniques by visiting the Portland Distict's Elk Creek Project and those North Pacific Division and Portland District personnel who are versed in RCC application.

3. Should further study of foundation conditions at the site reveal that a RCC gravity design is feasible, then the procedure outlined above should facilitate "changing course." In the interim, I see no value in proceeding with both design concepts to an equivalent level of detail. I recommend concentrating on the embankment dam concept and the investigation and definition of the complex foundation conditions.

FOR THE COMMANDER:

WILLIAM N. McCORMICK, JR.
Chief, Engineering Division
Directorate of Engineering and Construction

ATTACHMENT 1

SPDED-PC (DAEN-ECE-B/10 Oct 86) 1st End Mr. Krhoun/clm/6-5984
SUBJECT: Santa Ana River, CA

DA, South Pacific Division, Corps of Engineers, 630 Sansome Street, Room 720, San Francisco, CA 94117-2206

26 NOV 1986

TO: Commander, Los Angeles District, Attn: SPLED

1. During discussions at the General Design Conference (GDC) for Seven Oaks Dam on 28-30 October 1983 it was apparent that the issue of using roller compacted concrete (RCC) dam in lieu of an earth and rockfill dam required additional consideration. Since an RCC dam, if feasible, could produce possible cost savings a professional and objective assessment on the two alternatives must be made. This office fully supports USACE's requirements in the basic letter and furnishes additional guidance in the following paragraphs.

2. As agreed at the GDC, you are requested to prepare an engineering assessment that presents a comparison of the two types of structures at the Seven Oaks Dam site. The assessment should be accomplished in two phases. Phase I should contain the technical feasibility of constructing both dams based on the latest data on the geologic and seismic conditions of the site. It should also indicate the basis of each design and modifications required to meet local site conditions. If RCC is determined to be technically feasible Phase II should provide a cost comparison of both alternatives.

3. You are also requested to obtain views of expert consultants in all appropriate disciplines on the analysis. Your selection of proposed consultants should be coordinated with the appropriate SPD staff. Views of at least two structural experts experienced in concrete dam structures should be included in the assessment. Because of its background and knowledge of RCC structures the views of Portland District are to be specifically sought and its views incorporated in the assessment.

4. As agreed at our meeting with NPD and Portland District on 19 November 1986, a conference will be held on 5 December at the Elk Creek Project site in Oregon to discuss the RCC alternative. A complete briefing on Seven Oaks Dam site condition should be provided to Portland District to assure its complete understanding of the specific and unique characteristics of the site. Messrs Wanket, Parrillo, Siesen, Tanouye and Krhoun from this office will participate.

5. It is requested that Phase I of the assessment be completed by 31 December 1986. Phase II, if necessary, must be completed by 15 January 1987.

6. By 1 December 1986 you are requested to furnish this office an action plan to complete the assessment by 31 December 1986. This action plan should include; consultants to be contacted, involvement of SPD and Portland District and a schedule which will not adversely impact on completion of the GDM by August 1988 if RCC is a viable alternative . Progress of the assessment should be presented at the IPR scheduled for 1? December 1986.

ATTACHMENT 2

FACSIMILE HEADER SHEET
(ER 105-1-5)

FROM (Name)	OFFICE SYMBOL	TELEPHONE NO.	RELEASER'S SIGNATURE		
F. Krhoun	SPDED-PC	556-5984	F. Krhoun		
TO (Name)	OFFICE SYMBOL	TELEPHONE NO.	# PAGES	PRECEDENCE	DTG
Bob Koplin	SPLED	798-5470	3	ASAP	

SUBJECT

*U.S. Government Printing Office: 198?-???-???

7. Any questions on the above should be directed to Mr. Frank Krhoun at FTS 556-5984.

FOR THE COMMANDER:

A. E. WANKET
Chief, Engineering Division

CLARENCE R. ALLEN
1000 EAST CALIFORNIA BLVD., APT. 306
PASADENA, CALIFORNIA 91106

8 December 1984

Mr. Norman Arno
Chief, Engineering Division
Los Angeles District
Corps of Engineers
P.O. Box 2711
Los Angeles, California 90053

Dear Mr. Arno:

This letter will report my conclusions concerning the seismic hazard of the Upper Santa Ana River damsite, following my visit with Mr. Lukesh of your staff to the area on July 24th and our conference in Los Angeles on October 16th. Needless to say, these are somewhat preliminary opinions, inasmuch as my visit was very brief, very little detailed geologic work has as yet been done in the area, and I have not had the opportunity to examine aerial photographs of most of the reservoir area. My opinions are based primarily on (1) our brief site visit, (2) my prior knowledge of the region dating back to the time of my Ph.D. thesis work in 1950-54 (Allen, 1957), (3) my continuing studies of southern California seismicity and seismic hazard (e.g., Allen et al., 1965; Allen, 1975; Crook et al., in press), and (4) my continuing involvement in studies of major damsites elsewhere in the world, particularly with regard to the problem of reservoir-induced earthquakes (e.g., Allen, 1982a). In addition, as you are aware, I was involved in the earlier discussions concerning the nearby Mentone damsite (Allen et al., 1975a; Allen, 1981, 1982b).

It must be recognized at the outset, of course, that the proposed damsite is very close to the San Andreas fault--about one mile away--and that a great earthquake of about magnitude 8 must be assumed to be capable of occurring on the adjacent segment of this fault during the service life of the structure. Indeed, my colleague Professor Kerry Sieh, on the basis of his paleoseismological field work in the Cajon Pass and Salton Sea regions, rates this particular segment of the San Andreas fault as the most likely to break in association with a great earthquake in the near future--with a probability of 50% to 90% within the next 50 years (Sieh, 1984; Caltech press release, 24 Sept. 1984). Insofar as I am aware, the Corps of Engineers is fully cognizant of this situation and is prepared to design the dam accordingly--as was also the case for the earlier Mentone site. Additional considerations at the Upper Santa Ana River damsite to which you wished me to direct my attention were (1) the presence of possibly active subsidiary faults even closer to, or beneath, the proposed structure and its associated

ATTACHMENT 3

tunnels, and (2) the possibility of reservoir-induced earthquakes. In this connection, I will also comment below on further exploratory work that might help to resolve these concerns.

Although detailed geologic mapping by your staff has not yet commenced, two throughgoing faults close to the damsite have been identified that warrant concern. One of these, the Mill Creek fault (or North Branch of the San Andreas fault), passes about 3,000 feet north of the damsite and will cross a deep part of the reservoir. The preliminary geologic map prepared by your staff shows the Mill Creek fault in this area as lying beneath unbroken Pleistocene terrace gravels, which would imply that it is relatively inactive as compared to the nearby South Branch of the San Andreas fault, which multiply breaks much younger Holocene strata and produces a clear scarp and ground-water barrier in very young alluvium at the mouth of the canyon. A Caltech graduate student who has been doing geologic field work in this area, Ray Weldon, estimates (personal communication) that the age of this Pleistocene terrace is about 50,000 years. From what I could see in the field, I would tentatively agree that the terrace is unbroken, but one could and should look at this situation much more closely. However, even if the Mill Creek fault were considered to be active, the kind of shaking at the damsite that an earthquake on it might generate would be comparable to that which must already be considered from the nearby South Branch of the San Andreas fault, so that the question of the degree of activity of the Mill Creek fault is somewhat academic insofar as it affects the design of the proposed dam.

Traversing the downstream toe of the proposed dam is a second fault, informally termed the "Subsidiary fault" by your staff. It can be seen clearly in the west canyon wall, where it separates crystalline rocks of different types. Fortunately, it is apparently truncated here by an unbroken patch of Pleistocene terrace gravels about 120 feet above the canyon floor. This patch is a remnant of the same terrace as that which apparently truncates the Mill Creek fault farther upstream, so that this fault, too, can probably be considered relatively inactive--or at least not able in itself to generate a significant earthquake or sustain very large displacement.

As I understand the present plans, one concept being considered is a reservoir which would have a maximum water depth, with the Standard Project Flood and full conservation, of about 560 feet. This, however, would be a very temporary and rare situation, lasting for a few days or weeks at the most. The maximum sustained water depth, with full-time conservation storage, would be about 320 feet, which puts it in the category of a "deep" reservoir by worldwide standards (Packer at al., 1979; Stuart-Alexander, 1981; Baecher and Keeney, 1982). Therefore, in my opinion, there must be some concern for the possibility of significant reservoir-induced earthquakes, which was not the case for the Mentone damsite. Although the volume of water and debris to be stored at full conservation is about 100,000 acre feet, which is not large by world-wide standards, the correlation between induced earthquakes and water depth seems to be stronger than with volume, and even relatively small reservoirs with this depth of water have probably been associated with significant reservoir-induced earthquakes (Packer et al., 1979).

An added concern here is that the Mill Creek fault, which is a major geologic structure, traverses a deep part of the reservoir. Even if the Mill Creek fault is eventually deemed inactive, it clearly represents a through-

going zone of weakness, and the warm springs near the junction of Warm Springs Canyon and the Santa Ana River indicate that deep water circulation is currently taking place. Thus the Mill Creek fault, in my opinion, must be considered as the potential locus of a reservoir-induced earthquake (Allen, 1982), although the probability of this happening is very small, based on experience at other reservoirs. The maximum worldwide reservoir-induced earthquake to date was the 1967 magnitude 6.5 event at Koyna, India, and although still larger induced events are not physically impossible, planning for them seems unreasonable (Allen, 1982a). Thus the naturally occurring San Andreas earthquake would probably cause heavier shaking at the Upper Santa Ana River damsite than would any reasonable reservoir-induced earthquake, so that the possibility of a reservoir-induced event should not lead to additional design considerations except insofar as associated surface displacement on the Mill Creek fault might cause problems in itself.

However, the natural seismicity of this part of the San Bernardino Mountains is sufficiently high--with a 1935 earthquake of magnitude 5.1 having occurred perhaps on the Mill Creek fault nearby (Richter et al., 1958)--so that any earthquake in the vicinity of the reservoir might be claimed to be reservoir-induced and could not easily be proved otherwise. In all candor, we must admit that the state of the art is presently such that scientists would have difficulty in determining with confidence whether or not a given earthquake in the vicinity of the dam was in fact related in any way to the presence of the reservoir.

For the Mentone damsite, the Board recommended that 2 to 3 feet of fault displacement, in any direction, be considered credible on arbitrary subsidiary faults that might underlie the dam, because of its proximity to the San Andreas fault and because the local bedrock structure was concealed by alluvium (Allen et al., 1975a; Allen, 1981). The Upper Santa Ana River damsite is likewise close to the San Andreas fault, although the local bedrock is virtually 100% exposed. An added complication for the new site is that, because the San Andreas fault dips about 65° north at the mouth of the canyon, the site may be on the upthrown block of a thrust fault and is therefore in a somewhat more vulnerable location from the point of view of subsidiary faulting than are locations on the downthrown block to the south (Sherard et al., 1974). While it is true that the San Andreas fault probably steepens with depth, and it is certainly not a deep-seated thrust fault analogous, e.g., to the fault causing the 1971 San Fernando earthquake (Allen et al., 1975b), the San Bernardino Mountains may have been uplifted by a vertical component of displacement along the bounding San Andreas fault, and active splay and subsidiary faults appear to me more common north of the fault (within the mountains) than to the south. Furthermore, active strands of the San Andreas fault cannot be followed with complete continuity through the San Gorgonio Pass area to the east of the site, and it seems likely that a great earthquake in this region will be characterized by a certain amount of pervasive "shattering" of the southeastern San Bernardino mountain block, although predominantly along pre-existing faults and fractures.

A very recent and intriguing argument by Weldon and Humphreys (submitted for publication) and Weldon (1984) proposes that the relief along the south face of the San Bernardino Mountains, instead of being caused by a vertical component of displacement on the San Andreas fault, is caused simply by horizontal slicing apart by the San Andreas fault of a former combined San Bernardino-San Gabriel mountain mass. That is, the north-facing northern

escarpment of the San Gabriel Mountains is visualized as formerly juxtaposed against the south-facing southern escarpment of the San Bernardino Mountains, so that subsequent right-lateral strike-slip displacements on the San Andreas fault have simply sliced apart the two halves of the former single mountain mass without any significant vertical components of displacement. Supporting this point of view, according to Weldon and Humphreys, is the absence of evidence for current horizontal compression across the San Andreas fault in most of the San Bernardino-Cajon Pass area. While I agree that this argument is intriguing and probably at least in part correct, I am impressed with the northerly dip of the San Andreas fault at the mouth of the Santa Ana River Canyon, and I feel that a conservative approach demands that this be considered indicative of a vertical component of displacement. Furthermore, even Weldon and Humphreys agree that considerable current compressive stresses exist in the San Gorgonio Pass area to the southeast, where the fault traces are more complicated and where numerous active thrust faults are demonstrably present (Allen, 1957), and the Santa Ana Canyon area is sufficiently close to this area that it must share some of this same complex stress system.

With these factors in mind, I recommend designing the Upper Santa Ana River Dam on the assumption that as much as 4 feet of surficial fault displacement in any direction could take place arbitrarily beneath the facility, albeit as an exceedingly unlikely "maximum credible" event, and assuming about 300 feet of depth of permanent water storage. If the reservoir were instead to be nearly empty much of the time, a less conservative figure could perhaps be used. And if one could demonstrate that the local bedrock was completely unfractured, or that unbroken Pleistocene terraces covered the entire site area, this degree of conservatism might not be necessary, but such is not the case. As was recommended for the Mentone damsite (Allen, 1981), the specified fault displacement does not have to be assumed to pass directly through every localized structure of the dam (e.g., a spillway gate), but the basic integrity of the embankment and long tunnels--if critical to public safety--should be assured with this possible arbitrary displacement in mind.

In order to complete the initial seismic-safety analysis of the Upper Santa Ana River damsite, it is my opinion that several additional tasks should be undertaken by your staff:

(1) A high-quality geologic map of the entire reservoir area at 1:24,000 scale, emphasizing neotectonic relationships, should be produced, utilizing existing mapping but also undoubtedly requiring further field studies of your own or under contract. This is not a small task, but, in my opinion, experience at other reservoir sites, such as that at Auburn, demonstrates the wisdom of gaining a thorough understanding of the geological environment at an early stage in the project.

(2) As I mentioned to Mr. Lukesh in the field, it is essential that vertical aerial photographic coverage be obtained of the entire reservoir area for use in these studies. (Adequate coverage undoubtedly already exists at scales of about 1:20,000 and will not necessarily require a new flight contract.) Perhaps this has been accomplished since the time of my visit.

(3) A special investigation of the "Subsidiary fault" on the west wall of the Santa Ana River Canyon should be undertaken to document the apparent truncation of the fault by Pleistocene terrace gravels. This might require

some cleaning of the contact, but the exposure looks relatively good already, at least from a distance.

(4) The suggested inactivity of the Mill Creek fault should also be the subject of a special field study, again involving relationships with terrace gravels. Since the Mill Creek fault is of some length, this study should include investigations of the fault outside of the immediate reservoir area as well as within it. I wouldn't be too hasty in accepting the conclusion that the Mill Creek fault is inactive. Although, as stated above, the degree of activity of the Mill Creek fault is a somewhat academic question in terms of the actual dam design, it is essential to have a thorough understanding of the tectonics of the reservoir area, and this fault is a key element within that tectonic pattern.

(5) Consideration should be given to increasing the seismographic coverage in the area. The nearest telemetered station of the USGS-Caltech network to the site is currently about 6 miles south at Crafton Hills (34° 02.11' N, 117° 06.66' W). Other nearby stations are at Mill Creek, about 10 miles east (34° 05.48' N, 116' 56.18' W), and at Butler Peak, about 11 miles north (34° 15.43' N, 117° 00.29' N). The nearest station to the west or northwest is at Cedar Springs Dam, more than 19 miles away. Certainly several years before construction starts, one or more telemetered stations should be established in the immediate reservoir area.

(6) Although Ray Weldon, the Caltech graduate student who has been carrying out field work in this area, is now committed to the U. S. Geological Survey and therefore apparently cannot formally consult for the Corps of Engineers, I urge that you communicate with him about his ideas on the structure and geomorphology of the region. Since these studies were a part of his Ph.D. thesis at Caltech, he should have no hesitation in talking about them.

Very truly yours,

CR Allen

Clarence R. Allen

cc: Mr. David Lukesh

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DEPARTMENT OF THE ARMY
PORTLAND DISTRICT, CORPS OF ENGINEERS
P. O. BOX 2946
PORTLAND, OREGON 97208-2946

Reply to
Attention of:

NPPEN-GE 10 December 1986

SUBJECT: Santa Ana Project, Seven Oaks Dam, Concrete Gravity Dam Evaluation

TO: Commander, Los Angeles District
ATTN: SPLED-G

1. At the request of the Los Angeles District, the Portland District has completed reviewing the Concrete Gravity Dam Alternative being considered at the Seven Oaks dam site. The information summarized in this letter was presented to Los Angeles District and South Pacific Division personnel on 5 Dec 86 at the Elk Creek Project Office. Copies of the agenda and viewgraphs used during the presentation are enclosed.

2. This evaluation is based primarily on a review of the technical appendices to the Phase I General Design Memo; an Information Paper comparing a Roller Compacted Concrete (RCC) dam to an embankment dam (dated 28-30 Oct 86); and an RCC dam cost estimate (dated 28-30 Oct 86). The Portland District arrived at the following conclusions based upon a limited review of these materials and telephone conversations with various Los Angeles District staff members:

a. We agree with the conclusions Los Angeles District presented in their comparative study. Either a concrete gravity or embankment dam is technically feasible and could be designed and constructed at the Seven Oaks dam site. At this level of investigation, the costs for the two structures are reasonably comparable. Cost for the embankment alternative is estimated at $304 million, and for the concrete gravity dam alternative, $275 million.

b. Generally, the site does lend itself to an embankment dam design, however, insufficient technical information has been developed at this stage of investigations to conclude that:

(1) A properly designed concrete gravity dam is more vulnerable to "significant" damage than an embankment dam. All concrete dams crack, however, insufficient data is presented to determine if cracking would be of sufficient magnitude to endanger the structure.

(2) Utilization of the roller compacted concrete method of concrete placement would limit the maximum height of the dam. As pointed out in the comparative analysis, the highest RCC structure to date is

ATTACHMENT 4

approximately 300 feet high. Our experience leads us to believe that roller compacted concrete mixes can be designed to equal those of conventional mass or formed concrete however. Consequently, the Portland District has approached this evaluation on the basis of concrete gravity dam design without differentiating the actual manner of concrete placement.

c. Studies to date (through the Phase I GDM) are technically sufficient to support an embankment dam design, however, there are many uncertainties regarding the site's adequacy for concrete gravity dam construction. We believe that certain items need to be evaluated in greater detail before firm conclusions can be drawn, specifically, geotechnical considerations such as: seismic response of the foundation, foundation deformation, excavation depth, and sliding stability. In the structural area, the seismic response of the structure, concrete strength, and contraction joint requirements need to be evaluated in additional detail.

d. Analysis of the cost estimates for the two structures revealed the following items that need to be examined in greater detail.

(1) Embankment Dam. Approximately 81 percent of the entire cost of the project is connected with five groups of bid items: impervious borrow; filter, transition, and rock borrow; spillway rock excavation; impervious embankment; and filter, transition, and rock embankment. The scope of the project is very large. Approximately 37 million cubic yards (mcy) of embankment and excavation will be involved. Consequently, the total cost estimate is very sensitive to the unit costs assigned to each bid item. For instance, there are approximately 34 mcy of filter, transition, and rock embankment. Due to this large quantity, a variation in the unit cost of only $.10 per cy would affect the final cost estimate by $3.4 million. Consequently, it may be worthwhile to reevaluate all of the unit costs used in the cost estimates to determine whether they are sufficiently accurate to allow comparison of the two structures.

(2) Concrete Gravity Dam. Due to the large quantities involved in the concrete gravity dam, the same question holds true. A small variation in the unit cost will result in a large change in the project cost. Unit costs for the roller compacted concrete and other concrete may need to be reevaluated. One of the key factors affecting the concrete and gravity dam design costs is use of contraction joints. Contraction joints have been included in the concrete gravity dam design at approximately 50-foot intervals. Recommend that the need for all contraction joints be reevaluated. The installation of this number of joints will significantly impact the unit cost of the RCC because use of those joints will have a major impact on the contractor's ability to place RCC in an economical manner. If it is confirmed that all of the contraction joints are necessary due to the tectonic setting of the dam, the RCC price currently included in the cost estimate is likely to be low.

e. Suggest that Los Angeles District consider pursuing parallel design of both the concrete gravity and embankment dam until sufficiently

detailed technical analyses and cost data are developed to support or reject one of the alternatives. Sufficient time appears to be available prior to completion of the draft Phase II GDM scheduled for completion in April 1988.

f. The current level of Geotechnical investigations at the site is considered minimal, and the completion of parallel designs would not be a total duplication of effort since most work would be required for either structure. Suggest that Los Angeles District consider performing the following additional geotechnical explorations:

- Core drilling and detailed fracture logging.
- Large diameter explorations.
- More extensive materials testing.
- Borehole and core photography.
- In situ testing.

In addition, it is suggested that the following engineering analyses be performed as part of this parallel design effort:

- Bedrock fracture studies.
- Stability analyses.
- Finite element modeling.

Estimated cost for the additional geotechnical investigations and engineering analyses is approximately $250,000 to $500,000.

g. The design criteria for the project need to be firmly determined. Specifically, the normal operating pool level needs to be established. It is our opinion that for a low (100 to 200 feet) pool there are measures that could be employed to increase the safety of a concrete gravity dam to make it comparable to an embankment dam. One alternative would be to use an upstream filter/transition/crack stopper zone. We believe that this condition can be analyzed coincident with the design earthquake and provide for a safe, stable structure. If the normal operating pool is at spillway crest 500+ feet above the stream bottom, however, the technical problems associated with designing a structure of equal safety to the embankment dam become much more technically complex. Additional studies would be required.

h. The concrete gravity dam design is not presently at an equivalent design level with the embankment dam design. There are serious unanswered questions that would need to be resolved prior to switching from the embankment to the concrete gravity dam design. If the recommended investigations and analyses are done it may be possible to overcome these problems and design the concrete gravity dam. If no additional geotechnical investigations and engineering analyses are to be performed, however, and a decision must be made now on which direction to proceed in the future, then the earth embankment design is the preferred alternative.

NPPEN-GE 10 December 1986
SUBJECT: Santa Ana Project, Seven Oaks Dam, Concrete Gravity Dam Evaluation

3. We enjoyed meeting Los Angeles District and South Pacific Division staff members at Elk Creek, and believe this first meeting was very beneficial from your standpoint and ours. We look forward to working with you in the future on other challenging aspects of the Santa Ana Project.

FOR THE COMMANDER:

ROBERT P. FLANAGAN, P.E.
Chief, Engineering Division

2 Encl
1. Agenda
2. Viewgraphs (10)

CF:
NPDEN-G
SPDED-G
NPPEN-DB
NPPEN-PM

QUESTIONS

FOR

GEOTECHNICAL EXPERTS

SEVEN OAKS DAM ALTERNATIVES

A. Given the 4-foot postulated displacement and the condition of a full reservoir pool at the site of the Seven Oaks Dam:

1. Provide a generalized discussion of the anticipated performance of the zoned earth-rock embankment.

2. In your opinion are the preventive measures devised to mitigate damage for the RCC structure appropriate as it relates to

(a) the foundation contact?

(b) the abutment contacts?

(c) the structure?

3. If not what would you propose instead for

(a) the foundation contact?

(b) the abutment contacts?

(c) the structure?

4. Provide a generalized discussion of the remedial measures, if any, required for each type of structure as a consequence of the postulated displacement to re-establish safe functioning of dam.

B. Do you agree that for the RCC structure a reduction of the strength parameters of the foundation-concrete contact is necessary to account for the effects of separation due to displacement and if so by how much?

C. Do you agree or disagree with the Los Angeles District that the earth-rock embankment is the appropriate and prudent alternative for this site?

H. Bolton Seed, Inc.

623 Crossridge Terrace. Orinda. California 94563 (415) 254-3036

February 19, 1987

Norman Arno, Chief
Engineering Division
Department of the Army
Los Angeles District, Corps of Engineers
P. O. Box 2711
Los Angeles, CA 90053

Dear Mr. Arno,

In response to your letter of January 12, I have reviewed the technical paper entitled "Phase I Assessment, Embankment and Roller Compacted Concrete Dams" together with the Geotechnical Appendix to the Phase I General Design Memorandum. I have also received the Questions for Geotechnical Experts which you would like me to address relative to the Seven Oaks Dam Alternatives and my responses are presented below.

I might note first, however, that from a design point of view, a special feature of the Seven Oaks dam-site is the problems presented by its location adjacent to the San Andreas fault and the potentially very high seismicity of the area involved. There are ample precedents for construction of zoned earth dams or concrete gravity dams of the scale proposed in areas of low seismicity and there is precedent for roller compacted concrete dams up to 300 ft high in such areas. However from a seismic design point of view, the Seven Oaks site is as difficult and complex as virtually any site for which dams have been designed anywhere in the world. The San Andreas fault passes within a mile of the site with the Northern Branch of the fault crossing the reservoir area about 3000 ft north of the proposed dam-site and the Southern Branch passing

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about 5000 ft to the south of the site. The most recent fault activity seems to have been concentrated on the latter trace, but never-the-less the seismic geology of the area is extremely complex. The fault appears to be "locked" in this section and the potential for a major earthquake (Magnitude $\approx$ 8) occurring in the next 50 years is estimated to be about 50 to 90%. This is a formidable prospect and it has led to extremely stringent design criteria which may be summarized as follows:

1. Extremely strong ground shaking from a Magnitude 8± earthquake occurring within about half a mile from the site and producing ground motions as strong as any earthquake is likely to produce. Peak accelerations at the ground surface are estimated to be about 0.8g and the duration of strong shaking is likely to exceed about 60 seconds.
2. Potential fault-related movements at the dam-site which Professor Allen has discussed in his report and which lead him to the recommendation that the dam should be designed "on the assumption that as much as 4 ft of surficial fault displacement in any direction could take place arbitrarily beneath the facility, albeit as an exceedingly unlikely "maximum credible event...."

While I have no doubts concerning the ability of the profession to design earth, rockfill or concrete gravity dams to withstand the postulated level of shaking, there is very little field performance data to demonstrate this fact. The only field performance data of which I am aware for Magnitude 8± earthquakes, comes from the 1906 San Francisco earthquake (for which no ground motions were recorded) and the 1985 Mexico earthquake, where recorded ground motions in the vicinity of major dams (El Infiernillo and La Villita)

were unusually low (about 0.15 to 0.2g). In no case is there clear evidence of a dam withstanding major foundation displacements of the type described above however. The Lower Crystal Springs Dam, a curved gravity concrete structure about 155 ft high and located about 1300 ft from the San Andreas fault suffered no damage in the 1906 San Francisco earthquake but there does not appear to have been any major foundation displacements in this event at the Crystal Springs site. For other concrete gravity structures located very close to faults triggering earthquakes, the maximum earthquake to which they have been subjected appears to be about Magnitude 6.5. The Pacoima concrete arch dam survived without major damage the 1971 San Fernando earthquake of 1971, which probably produced ground accelerations near the base of the dam with a peak acceleration close to 0.8g and shaking lasting for about 12 seconds; there was a small opening of a joint between the arch and thrust block near the top of the dam but this did not present problems at the time. Again however it does not appear that there was any severe distortion in the supporting rock formations comparable to a 4 ft surficial displacement in any direction.

The same is true in large measure for embankment dam performance. Many such dams are known to have performed well in nearby moderate earthquakes with Magnitudes up to M = 6.5 or so, and the Hebgen Dam satisfactorily withstood (albeit with some damage) the effects of a Magnitude 7.1 earthquake occurring on a fault located about 1000 ft from the dam-site. No ground motions were recorded, however, and the extent of foundation deformation is not known. The largest body of performance data for earth dams during very large earthquakes (Magnitude 8±) seems to be that provided by the observed performance of a number of earth dams in the 1906 San Francisco earthquake.

In this event there were apparently four dams located very close to the San Andreas fault--the San Andreas Dam, the Upper Howell Dam, the Lower Howell Dam and the Lake Ranch Dam. For the latter two dams the fault is reported to have intersected the dam and for the San Andreas Dam, there was reportedly a 7 ft horizontal fault displacement in the abutment. There was no significant damage in any of these cases, nor was there any significant damage to 6 other dams located between 1 and 2 miles from the fault. The accelerations to which these dams were exposed in this event can be expected to be very high, but there are no records available and there is no data on the possible extent of foundation displacements.

Much of the available information on the performance of concrete and embankment dams during earthquakes up to about 1977 is summarized in the following two reports:

"Response of Concrete Dams to Earthquakes" by K. D. Hansen and L. H. Roehn, Portland Cement Association, Denver.

"The Performance of Earth Dams During Earthquakes," by H. Bolton Seed, Faiz I. Makdisi and P. De Alba, EERC Report No. UCB/EERC-77-20, University of California, Berkeley, August.

My main purpose in reviewing the above information is to note that while there is some performance data to show that well-designed earth and concrete gravity dams have withstood the effects of strong earthquake shaking, there is little definitive evidence to indicate how either type of dam has performed under conditions of severe foundation displacements or distortions. In consequence the design criterion for potential foundation displacements at the Seven Oaks site becomes a dominant criterion in the design of any structure at this site.

In the light of the above comments, I conclude that the available information and the considerations involved in developing designs for zoned earth dams and roller compacted concrete dams at the Seven Oaks site are well-presented in the technical paper prepared by the Los Angeles District, and in the report on this paper prepared by the Portland District on the subject "Santa Ana Project, Seven Oaks Dam, Concrete Gravity Dam Evaluation," dated 10, December, 1986. Virtually all of the relevant facts seem to be presented in these two documents.

Other aspects of the design problems are addressed in my responses to your specific questions in the following pages.

Question A

Given the 4-foot postulated displacement and the condition of a full reservoir pool at the site of the Seven Oaks Dam:

1. Provide a generalized discussion of the anticipated performance of the zoned earth-rock embankment.
2. In your opinion are the preventive measures devised to mitigate damage for the RCC structure appropriate as it relates to
   (a) the foundation contact?
   (b) the abutment contacts?
   (c) the structure?
3. If not what would you propose instead for
   (a) the foundation contact?
   (b) the abutment contacts?
   (c) the structure?

4. Provide a generalized discussion of the remedial measures, if any, required for each type of structure as a consequence of the postulated displacement to re-establish safe functioning of dam.

<u>Response to Question A</u>

1. Given the 4-ft postulated displacement and the condition of a full-reservoir pool at the site of the Seven Oaks Dam, I believe that a zoned earthrock embankment with an appropriate number of defensive design measures as described in the LA District technical paper, would provide a satisfactory level of performance in the event of a major earthquake. I assume that there could be a single fault off-set under the embankment or there could be a series of arbitrary movements leading to an undulating rock distortion under the structure. Either type of movement could produce deformations within the embankment leading to a tendency for cracking of the impervious core. However with appropriate selection and compaction, core material would hopefully be placed with sufficient plasticity to minimize the cracking, and with appropriate and wide filters provided downstream of the core and appropriate "crack-stopper" zones provided upstream, any cracks in the core or joint openings in the foundation would be self-sealing and the resulting leakage would not be harmful to the stability of the dam. The critical elements of the design are (a) the ability to provide a relatively deformable core, widened at the base of the core to provide a broad contact zone and (b) the ability to provide both a filter system and a self-healing crack-filling system within the embankment.

2. The problem becomes more difficult with an RCC structure because (a) the material itself is much more brittle and thus cracks and openings are likely to be larger than would occur in a plastic clay core and (b) the foundation displacements may disrupt the foundation drainage system and lead to unacceptably high pore pressures along the base of the dam, or even to a continuous opening under the dam from the upstream to the downstream sides. The measures proposed to handle the resulting seepage problem if openings were to develop along the base of the dam are essentially the same as those applying to the embankment dam design. In effect, in the lower part of the dam (up to El. 2200), the RCC dam is protected by a zoned embankment dam. This I believe to be the best defensive measure to seal any openings which might develop at the base of the dam. However it provides no protection against leakage through openings that may develop under the upper part of the structure between about El. 2200 and 2520, and if water had direct access to a continuous opening in this zone, it could cause progressive erosion and possibly lead to failure.

3. In my view also, the proposed extent of the upstream embankment zone at the abutment contacts is not sufficiently large to provide the required level of assurance for sealing the openings that may develop either in the structure or at the structure-foundation contact. I do not know any way to seal cracks and openings that may develop in dams or at their foundation contacts which has more potential for self-remediation than the provision of "crack-stopper systems" of the type proposed, especially in association with downstream filter systems. The latter can not be provided with the RCC structure to the same extent as they are in a zoned

earth embankment. However adequate protection for an RCC dam would require a much larger upstream buttress than is currently being proposed.

4. Following the occurrence of the postulated earthquake and the possible anticipated level of damage which might occur to either a zoned earth embankment or a RCC dam, I would expect that necessary remedial measures would involve the following:

   (a) <u>Zoned earth-rock embankment</u>

   Possibly a rapid lowering of the reservoir water level to a safe elevation followed by limited grouting of the core and core/foundation contacts as seems appropriate (based on seepage observations and pore pressure measurements) and some minor re-shaping of the upstream embankment if any movement of shell material has occurred.

   (b) <u>RCC Dam</u>

   Rapid lowering of the reservoir level to a safe elevation followed by grouting as necessary to seal openings at the dam/foundation contact or cracks in the dam itself, possible replacement of water stops or repair of joints as found to be necessary, and possible re-establishment of drainage system if it is disrupted.

I believe that if the postulated foundation displacements should occur, the repair of damage to the RCC dam would be far more extensive than for a zoned embankment dam.

Question B

Do you agree that for the RCC structure, a reduction of the strength parameters of the foundation-concrete contact is necessary to account for the effects of separation due to displacement, and if so, by how much?

Response to Question B

Since the postulated foundation displacement may involve an irregular deformation pattern across the base of the dam, leading to zones where there is loss of contact between the base of the dam and the foundation rock, I believe that it is appropriate to reduce the strength parameters to allow for this effect. In particular I would suggest a significant reduction in the 'c'-component of the assigned strength parameters.

I would need to know more details of the basis for the present assignment of strength parameters and the analysis procedures with which they are associated before I could make a meaningful recommendation concerning the design parameters to be used.

However in addition to evaluating a factor of safety against sliding I believe it would be desirable to make a dynamic displacement analysis for the dam, of the type proposed by Newmark, to evaluate deformations which may occur during the period of earthquake shaking.

Question C

Do you agree or disagree with the Los Angeles District that the earth-rock embankment is the appropriate and prudent alternative for this site?

Response to Question C

It is clear from the preceding discussion that defensive measures are required in any type of dam constructed at the Seven Oaks site to mitigate against the effects of the postulated foundation displacements. I favor the selection of a zoned earth embankment at this site for the following reasons:

1. The flexibility of a zoned earth embankment, involving the deformability of the core and the inability of cohesionless zones to develop cracks make it easier to avoid cracking due either to foundation movements or earthquake shaking, than would be the case for a roller compacted concrete dam.
2. It is easier to construct a variety of defensive design measures in the embankment dam than in the RCC dam, to protect against the effects of earthquake shaking and foundation displacements. In the case of the RCC dam this is essentially limited to the provision of a "crack-stopper" or "space-filler" provision, but with an embankment dam it can also include the provision of filters and other measures discussed in the technical paper.

I am not aware that any defensive design measures specifically installed to protect against fault or fault-related displacements have been tested by actual earthquake-induced deformations, although they have been used in the belief that they would be effective if this should be the case. Until direct experience is obtained, however, some uncertainty inevitably exists, whether such measures be incorporated in a zoned earth-rock embankment or an RCC dam. Hence the choice of dam-type depends to some extent on the level of confidence generated by the proposed measures and this will inevitably

depend on the individual judgment of the engineers involved. I personally believe that a zoned earth-rock dam would be significantly less vulnerable to damage due to foundation displacements, and more amenable to protection against such displacements, than would an RCC dam, and that there is more evidence to support this position for embankment dams than for RCC dams.

3. Another factor of some importance in choosing between the different types of dams is the available experience in constructing these types of dams at other sites. Many embankment dams with central clay cores have been constructed to heights of 500 to 600 ft and shown to work very well. In California the prime example is Oroville Dam with a height of about 750 ft. Thus there is no extension of existing experience required in designing and constructing a 550 ft high dam of this type at the Seven Oaks site. However available experience with RCC dams is currently limited to dams about 300 ft high. It does not seem prudent to make the jump from 300 ft to 600 ft at one of the most difficult sites imaginable from a seismic point of view.

   Progress in dam design has generally involved the construction of dams with much smaller jumps in scale to ensure that no unanticipated problems develop as heights are progressively increased. With RCC dams this may involve the construction of a dam about 450 ft high at a good site, rather than attempting to make both a major jump in scale (from 300 to 600 ft high) and in complexity of site seismic geologic conditions at the same time.

The seismic problems at the Seven Oaks site are about as difficult as might be encountered at most sites likely to be selected for dam construction anywhere in the world. It would be a major accomplishment to design and construct a dam with which the profession is very comfortable at this site. There is also precedent for such a choice in recent years e.g. Tarbela Dam. It would be considerably more difficult to develop the same level of comfort with a relatively new type of dam with which we have limited experience and which has never been built to the required height, at the Seven Oaks site.

I also believe that this would be a major consideration from the point of view of the people living downstream. While public reaction is very difficult to judge, it is an important consideration in the light of public opposition to other critical structures in California and the impact of that opposition on the design and construction of such structures.

For these reasons I agree with the Los Angeles District that the earth-rock embankment dam is the appropriate and prudent choice for the Seven Oaks dam-site, even if the cost of such a structure is somewhat higher than that of an RCC dam.

Sincerely yours,

H. Bolton Seed

H. Bolton Seed

HBS/nh

RBP

RALPH B. PECK CIVIL ENGINEER: GEOTECHNICS

5 February 1987
J1017

Mr. Larry Lauro
Chief, Foundation and Materials Branch
Los Angeles District
Corps of Engineers
P. O. Box 2711
Los Angeles CA 90053

Dear Mr. Lauro:

This will acknowledge receipt of the following documents concerning the Upper Santa Ana River Flood Storage Project: "Volume 1, Appendixes A-E of the Supplement to Phase 1 GDM"; "Information Paper, Roller Compacted Concrete vs. Embankment, Comparative Study, Seven Oaks Dam, 28-30 October 1986"; and "Phase I Assessment, Embankment and Roller Compacted Concrete Dams, Seven Oaks Dam", 31 December 1986.

I have reviewed those parts of the documents dealing with the geotechnics, the seismicity, and the design of the embankment and roller compacted concrete alternatives. The purpose of my review was to assess the ability of the two types of dams to withstand the postulated earthquake movements.

The principal design requirement with respect to seismicity is the ability to cope safely with a displacement in the foundation or abutments of 4 ft in any direction, as recommended by Dr. Clarence Allen. The following comments are organized in accordance with the Questions for Geotechnical Experts furnished with the document of 31 December 1986.

A1. <u>Anticipated performance of Zoned Earth-Rock Embankment</u>

In my judgment, an embankment dam designed along the principles described in the documents, consisting of an

ATTACHMENT 6

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impervious core protected by generously dimensioned filter and transition zones of rounded particles and supported by rockfill with conservative external slopes, has the capability of withstanding the postulated movements without such damage that catastrophic loss of the reservoir would occur. Internal displacements on the order of 4 ft would cause readjustment and movement of the embankment materials, but the cohesionless and non-interlocking character of the filters and transitions would ensure that no voids in these materials could remain open. Although a throughgoing crack in the core is conceivable, enlargement of the crack by erosion would be limited by the action of the downstream filter, and the crack would be occupied by material washed in from the upstream filter. These principles have been utilized in the design of other dams under similar circumstances, including Los Angeles Dam.

A displacement of up to 4 ft by faulting does not imply the formation of an open crack of that width in the foundation rock, inasmuch as faulting is associated primarily with shearing. However, the opening of narrower fissures in the foundation is possible. The presence in the embankment dam of a substantial section of granular materials containing cobbles and boulders up to 3 ft in diameter ensures that the fissures would be bridged over by large particles that, in turn, would restrain successively smaller particles. The restraint to flow caused by the filter thus formed could be expected to reduce the velocity of flow sufficiently to resist massive erosion of the rock alongside the fissures.

A2. Preventive Measures for RCC Dam

The postulated 4-ft displacement in foundation or abutment could, among other possibilities, create a tunnel-shaped opening from upstream to downstream between the concrete and the underlying foundation rock, or a vertical break or

separation of blocks also producing an opening from upstream to downstream. Water having unimpeded access to these passages would escape under high head with great erosive ability. Although much of the foundation rock appears to be competent, erosion at the velocities associated with full pool behind such a high dam could be expected to enlarge the passage by attacking fault gauge, shear zones, blocks isolated by fractures, and locally weathered zones. Even with extensive and careful foundation treatment, rapid disintegration of weak portions of the foundation could not be precluded. The damage that has occurred to structures such as concrete-lined spillway tunnels attests to the erosive capacity of water under conditions of similar head and potential velocity.

The upstream foundation buttress fill is intended to mitigate the erosion by requiring the reservoir water to pass through a succession of flow-retarding materials, having the characteristics of filters and crack stoppers, in a manner analogous to the behávior of the transitions and filters in the zoned embankment dam. However, the protection is highly vulnerable, especially in the event of a vertical throughgoing opening. With full pool, water will have free access to the opening at all points above the top of the buttress fill, can enter the break at high velocity, can descend to the bedrock level, and can then initiate the destructive erosion previously mentioned. If the opening should not occur at a planned construction joint, the foregoing scenario would be probable. If the opening should occur at a joint with a water stop, the stop would be required to function without fail. Although an elaborate water stop has been designed for comparable movements for at least one dam in Latin America, neither it nor the suggested design for the Seven Oaks Dam has been tested by experience. The detail shown for the latter does not

include the connections to the bedrock where extreme distortions could be expected and where failure of the entire waterstop might be initiated.

In my view, reliance on an unprecedented and untried waterstop would not be prudent. Moreover, although construction of a zone containing rounded mobile cobbles to plug a crack or fissure with a width up to about 6 in. is practical and such a zone is within precedent, it is by no means certain that well graded rounded materials containing boulders up to several feet in diameter could be obtained and placed effectively to ensure the buildup of a filter upstream of say, a 4-ft opening. With a fault displacement of this magnitude, such an opening is conceivable in or beneath a concrete structure, as illustrated schematically in Plate 6 of the document of 31 December 1986, but would not be possible in or beneath a flexible embankment structure.

A3. Proposals for Improving RCC Structure

The buttress fill detailed on Plate 5 of the document of 31 December 1986 appears to be designed to protect only the foundation. If a throughgoing opening should develop in the concrete structure itself, the waterstop would provide the sole defense above the top of the Rock II zone. If the waterstop were ineffective, all the finer crest materials above the Rock II zone would wash through the dam and would only add to the erosive power of the escaping water. Complete protection would require carrying the Rock II zone up to a full pool level (a measure still subject to the uncertainty discussed in the preceding paragraph). Such a measure would be equivalent to constructing half of an embankment dam to safeguard the concrete structure.

The foregoing comments indicate that I do not envision any dependable means for providing safety of a concrete

structure comparable to that of a conservatively designed embankment dam under the postulated conditions.

A4. Remedial Measures Subsequent to Faulting

The embankment dam would suffer loss of material associated with the development of the bridging and filter action above a newly developed fissure in the bedrock. A depression, perhaps sufficiently localized to be called a sinkhole, might appear on the upstream slope. The dimensions could be determined by sounding, if below pool level, and an estimate could be made of the possible displacements near the rock surface and of the extent to which the zoning may have been disrupted. It is unlikely that any repairs would be needed other than placing graded material in the depression, without necessarily draining the reservoir, and restoring any lost freeboard.

If the fault should disrupt the outlet tunnel or if the control tower should be damaged, it might become difficult or impractical to empty the reservoir. Consideration might be given to providing redundancy in the outlet works.

Repairs to the RCC dam would depend on the nature of the damage. The drainage beneath a block that may have moved even a small distance may be disrupted. If so, stability studies might dictate restoration of the drains. The outlet works could be severed, possibly in the RCC section itself, but more likely within the buttress fill or at its juncture with the concrete. If the control works were located within the concrete section, evacuation of the reservoir could probably be accomplished even if severance occurred.

Damage to a waterstop would be difficult to repair and possibly impractical without emptying the reservoir. The critical zone at foundation level would not only be difficult to dewater but would probably be obstructed by debris from the buttress. If erosion had created cavities beneath the blocks

of the dam, filling the voids might also require unwatering and would, at best, be an expensive and time-consuming operation.

B. Reduction of Strength Parameters

Reduction of frictional resistance at the contact between concrete and rock to account for separation along this interface seems less logical than assuming a conservative uplift pressure and utilizing normal frictional resistances. If, instead of by separation, the blocks reach their final position by sliding, the cohesion component of sliding resistance might be reduced. Since this "cohesion" is actually an allowance for shearing or overriding asperities or irregularities, its magnitude is usually determined for design more by convention than by direct evidence. The lack of experience in dealing with blocks after sliding indicates a corresponding lack of ability to rationalize an appropriate reduction.

C. Choice of Alternative

The foregoing comments indicate that the RCC alternative would be ill suited to cope with the displacements postulated at the Seven Oaks site. It involves, among other features, dependence on waterstops under unprecedented conditions. The buttress fill would be vulnerable if a waterstop should fail, would not ensure safety unless extended to full pool, and even then would involve a cobble-boulder zone with constituents of exceptional size to plug an opening that might be expected as a result of the postulated fault displacement.

On the other hand, although the fault displacement is large, the width of opening in bedrock is expected to be significantly less, capable of being bridged by materials often used in embankment dams. Moreover, the ability of such a dam to survive does not depend on the integrity of a water barrier of unprecedented dimensions used under highly indeterminate deformation conditions.

I agree fully that the earth-rock embankment is the appropriate alternative. Indeed, under the severe requirement to withstand a 4-ft fault displacement in the rock, I do not believe that the integrity of the RCC alternative at this site can be considered to be established.

Yours very sincerely,

Ralph B. Peck

Ralph B. Peck

RBP/ajj

QUESTIONS
CONCRETE GRAVITY DAM
STRUCTURAL EXPERT

1. Provide a generalized discussion of anticipated behavior of the dam for seismic considerations provided in the Phase I assessment.

2. Vertical contraction joints at 50-foot spacing are provided in the concrete gravity dam design to minimize cracking and control the location movement within the structure due to postulated foundation displacement. Cost and impact on the construction are significant. A required spacing of about 200 to 300 feet is anticipated for control of thermal contraction. In your opinion, will the 50-foot joint spacing be effective for the intended purpose? Will a wider spacing be adequate?

3. Based on response to the previous question, can random cracking within the structure for a "worst case" displacement scenario provide a significant flow path through the concrete structure? Is there a defensive measure that can be incorporated in the design for random cracking?

4. Provide a generalized discussion of the remedial measures, if any, required as a consequence of the postulated displacement to reestablish the safe functioning of the dam.

5. Provide comments on the proposed defensive measures for the RCC dam.

6. Are there defensive measures that should be substituted for the ones proposed?

7. Should additional defensive measures be considered?

8. Foundation-concrete contact strength parameters are to be reduced in future analysis to account for effects of separation due to displacements. Provide views for establishing the amount of reduction from "undisturbed" values, establishing the areal extent that needs to be considered and the possible analysis techniques.

9. In your opinion, can a concrete gravity dam be designed for the site conditions with a high degree of assurance of satisfactory performance for seismic considerations and a full reservoir pool?

George W. Housner
1201 E. California Blvd
Pasadena, California 91011

Mr. Martin Carlassare 22 March 1987
Los Angeles District Office
Corps of Engineers
Los Angeles, CA

Dear Mr. Carlassare:

The following is my report on the seismic safety of a proposed roller compacted concrete dam at the Seven Oaks Dam Site in upper Santa Ana Canyon. A one day visit was made to the site with M. Carlassare and others of the District Office. The following documents have been reviewed:

1. Santa Ana River Basin- Seven Oaks Dam
   - Phase 1 Assessment, 31 Dec 1986
   - Phase 2 Information Paper- Roller compacted Concrete vs Embankment Comparative Study, 28-30 Oct.86
   - Phase 2 Supplemental Informatiuon Paper
2. Upper Santa Ana River Flood-Storage Alternatives Study- Appendices vol 1, Dec 1985
3. Report of Clarence R. Allen on seismic hazard of the upper Santa Ana River Dam Site, Dec. 8, 1984

Two discussions were had with C.R. Allen to clarify some items in his report and to obtain further insight into the nature and likelihood of the foundation displacements set forth in his report.

Nine questions related to the roller compacted concrete dam considered for the Seven Oaks site have been posed. These questions and my responses are given below:

ATTACHMENT 7

1. Question. Provide a generalized discussion of anticipated behavior of the dam for seismic considerations provided in the Phase 1 assessment.

There is a high probability that the site will experience strong earthquake ground shaking during the life-time of the dam and these seismic inputs would produce significant seismic stresses in the dam structure. The proposed design earthquake ground accelerations are conservative, in my opinion. It is also my opinion that the rolled concrete dam could be designed to withstand safely these ground motions, including the effect of full reservoir. The dam site is located between the North Branch and the South Branch of the San Andreas Fault- 3,000 ft from the North Branch and about twice as far from the South Branch. The proximity of the site to active faults raises questions as to the possibility of fault-type displacements occuring in the foundation beneath the dam in the event of a magnitude 8 earthquake on the adjacent San Andreas Fault system. The performance of a concrete dam that is subjected to fault displacements in its foundation is difficult to predict if the precise nature of the displacements are not known. If the amplitude of displacement were less than 1 foot, it is my opinion that the dam could be designed and constructed with remedial measures could enable it to survive without uncontrolled release of reservoir storage. If the permanent displacements in the dam foundation are large, then it is questionable if the dam could survive without uncontrolled release of water.

2. Question. Vertical contraction joints at 50-foot spacing are provided in the concrete gravity dam design to minimize cracking and to control the location of the movement within the structure due to postulated foundation displacement. Cost and impact on the structure are significant. A required spacing of about 200 to 300 feet is anticipated for control of thermal contraction. In your opinion, will the 50-foot joint spacing be effective for the intended purpose? Will a wider spacing be adequate?

The 50-foot spacing of the contraction joints would have the advantage of controlling the cracking and movement due to foundation displacement. In my opinion it would be somewhat easier to design the dam to be safe with a 50-foot spacing of the joints instead of a 200 feet, or more, spacing, providing that the amplitude of the foundation displacements was not too large. In my opinion, the 200 feet spacing of joints could accomplish the same purpose, providing the displacements were not too large, but additional engineering would be required for remedial measures.

3. Question. Based on response to the previous question, can random cracking within the structure for a "worst case" displacement scenario provide a significant flow path through the concrete structure? Is there a defensive measure that can be incorporated in the design for random cracking?

In my opinion, an unplanned and unexpected "random cracking" in the event of a large displacement in the foundation could provide a significant flow path through (or beneath) the structure. It is my opinion that defensive measures could be incorporated successfully if the foundation displacements were sufficiently small, but if the displacement were to be large no adequate defensive measures are foreseen.

4. Question. Provide a generalized discussion of the remedial measures, if any, required as a consequence of the postulated displacement to reestablish the safe functioning of the dam.

The postulated design displacement recommended by C.R. Allen is four feet in any direction at any location under the dam. In a "worst case" scenario, with 50-foot spacing of contraction joints, a rather large flow path could be opened under the uplifted monolith. A defensive measure that has been examined is the construtlon of an embankment against the upstream face of the dam. In the limit this could be an embankment of the same height as the

concrete dam. The complete structure would then consist of a dam whose upstream portion would be the same as an earth dam and the downstream portion would be concrete. If now a large flow path developed in the concrete portion and the foudation displacement deformed the adjacent embankment material and the reservoir was full, then there would be no guarantee that the flow of water could be controlled. In my opinion there is no 100% reliable and cost effective remedial measure for the postulated foundation displacements.

5. Question. Provide comments on the proposed defensive measures for the RCC dam.

The proposed defensive measure is to provide an embankment against the upstream face of the RCC dam but extending only part way up the face. The same comments apply as were made in response to Question 4. There is no guarantee that in the "worst case" scenario that uncontrolled release of a full reservoir would not occur.

6. Question. Are there defensive measures that should be substituted for the ones proposed?

No completely adequate defensive measures are available, in my opinion, for arbitrary 4 feet displacement in the foundation of the dam.

7. Question. Should additional defensive measures be considered?

No adequate defensive measures come to mind.

8. Question. Foundation-concrete contact strength parameters are to be reduced in future analysis to account for the effects of separation due to displacements. Provide views for establishing the amount of reduction from "undisturbed" values, establishing the areal extent that needs to be considered and the possible analysis techniques.

The postulated displacements could reduce the cohesion between dam and foundation and thus reduce the resistance to sliding. An in-depth analysis of this situation should be made if the RCC dam remains a viable consideration.

9. Question. In your opinion, can a concrete gravity dam be designed for the site conditions with a high degree of assurance of satisfactory performance for seismic considerations and a full reservoir pool?

In my opinion an RCC dam can not be designed for 4-foot foundation displacement with a high degree of assurance of satisfactory performance (at reasonable cost). In discussion with C.R. Allen, he stated that he thought that an in-depth geological investigation of the site would likely provide evidence that would warrant reducing the 4-foot design displacement but not below one foot. In my opinion the Seven Oaks dam site has almost maximum seismic exposure, exceeded only by a site located directly on the San Andreas fault. In view of this, I would not recommend building an RCC dam on the Seven Oaks site if there is not a high degree of assurance that it will perform satisfactorily in the "worst case" scenario; this would require a relatively small design displacenent in the foundation.

George W. Housner

George W. Housner
Consultant

GWH/bms

LRA

LINDVALL, RICHTER & ASSOCIATES
EARTHQUAKE SCIENCES & ENGINEERING

PRINCIPALS

Frederick C Lindvall
Charles F Richter (1900-1985)
C. Eric Lindvall
Roy W Clough
Ronald F Scott
J Brent Hoerner

March 30, 1987

SEVEN OAKS DAM SITE

In response to your request, I am pleased to give my comments on the roller compacted concrete dam that has been proposed to be built in the Santa Ana River Basin at the Seven Oaks damsite, and to respond to your 9 questions about the expected seismic performance of the proposed structure. My contact with this project started with the visit to the site on February 6, 1987 in company with Eric Lindvall of LRA, Jim Tonaya of Corps of Engineers, San Francisco, and you.

Subsequently I arranged a meeting with George Housner which was held on March 9, 1987 at the California Institute of Technology; during that visit we spent a couple of hours in discussion with Clarence Allen to get more background on his suggestion that the Seven Oaks Dam should be designed for 4 feet of surficial displacement directly beneath the dam. It also may be appropriate to mention that my experience with the threat of possible fault displacement beneath a concrete dam included my service with the Auburn Dam Consultants Board appointed by the U.S. Bureau of Reclamation during the design of that dam. The suggestion of a possible sympathetic fault break beneath the dam was a major factor in the seismic safety evaluation of that proposed structure.

Before responding to your specific questions on the expected dam performance, I will make some comments on the suggested fault break design requirement. Of course I am not a geologist or seismologist, but I believe that this postulated fault displacement should be evaluated carefully before it is accepted as a definite design requirement; any possible reduction of the arbitrarily specified four foot movement could have a major influence on the expected dam performance.

825 COLORADO BOULEVARD, LOS ANGELES, CALIFORNIA 90041 • (213) 254-5257

ATTACHMENT 8

First it must be recognized that the seismic environment at the dam site is dominated by the San Andreas fault located at a distance of about one mile, and that the design of the dam for inertial forces will be controlled by a magnitude 8+ earthquake at the closest segment of the fault. Also it is reasonable to assume that such an earthquake has a 50 to 90 percent probability of occurring during the next 50 years; so this would be considered as the Design Earthquake as well as the Maximum Credible Earthquake in the design of the dam for ground shaking excitation.

However, the probability of occurrence of fault displacement beneath the dam is an entirely different matter. It is important to note that earthquakes with significant fault displacements are believed to have taken place regularly in the adjacent segment of the San Andreas fault at time intervals of 150 to 200 years; thus it can be assumed that 6 or 7 such earthquakes have occurred in the past 1000 years. Yet there is no evidence that any sympathetic fault break or shattering have occurred during that time in the foundation rock that will underlie the proposed dam. So it is clear that the postulated rupture beneath the dam has much lower probability of occurrence than do earthquakes on the San Andreas. Moreover, as mentioned in Clarence Allen's letter, the terraces overlying the Mill Creek fault and the subsidiary fault do not appear to be broken. So there is no direct evidence to support the hypothesis of fault displacement beneath the dam, and it must be accepted that the probability of such occurrence is at least two orders of magnitude less than that of the San Andreas earthquakes.

For these reasons I have considerable doubt about the validity of the fault displacement criterion that has been proposed for this dam, especially specifying the arbitrary amplitude of four feet. It must be accepted that shattering and distortion of the upthrown block would be possible if there were a major thrust movement of the San Andreas fault. But there is little evidence to indicate that thrust movements have occurred at this segment of the San Andreas, and the possibility of four foot sympathetic displacements in the rock at the base of the dam seems inconceivable. It would be much easier to imagine displacement of up to one foot, and the performance of the dam in such a case probably would be much better.

Another factor that must be established definitely before the seismic safety of the dam can be evaluated is its proposed use. The project has been described to me in the context of flood control, and that type of usage poses for less seismic hazard than if the dam were intended for irrigation, power production or recreation, all of which require storing a significant body of

water in the reservoir. In operation for flood control, the reservoir could be maintained at a low level which would present no hazard to the population downstream. The probability of an earthquake that produces fault displacement beneath the dam occurring during the very limited intervals of time when the reservoir would be full is so remote as to be completely negligible.

Because of my extreme skepticism about the worst case scenario that has been postulated for this dam, I find it very difficult to respond directly to the questions that have been posed. Moreover, if the design is required to maintain a full pool while accommodating the proposed four foot displacement, it is hard to see how a concrete dam can be made cost effective in comparison with an embankment dam. As a matter of fact, I doubt that the safety of an embankment dam could be assured during a four foot base displacement applied in an arbitrary direction while maintaining a full pool, and I expect the full pool requirement would be abandoned if such displacements must be accommodated.

Thus in my opinion, the answers to the nine questions are almost immaterial. However, I will comment on each question in turn.

1. The most important aspect of the fault displacement mechanism to be considered beneath the dam is that the basic fault rupture occurs at some significant depth, and it then propagates to the surface <u>if</u> there is no rigid structure such as a concrete dam to inhibit its surface movement. The rock through which the fault propagates is a jointed fractured material, whereas the dam at the surface is a solid, homogeneous mass; thus there would be a major tendency for the joints in the rock to accommodate the potential fault displacement rather than for the rock to force the displacement into the concrete. This tendency exists for any hypothesized "free field" fault movement, and in my judgment a one foot free field displacement probably would be fully resisted without cracking the dam. Of course, a much larger free fault input would be likely to produce some fracture of the dam, but with lesser crack displacement than the hypothesized free field movement.

2. Vertical contraction joints effectively increase the deformability of the dam; thus its resistance to the free field motions would be reduced and greater crack or joint openings could be expected in the jointed dam. In my opinion the joints should be spaced as far apart as thermal conditions permit so as to take advantage of the beneficial effects of the dam crack resistance.

3. It is possible that random cracking of significant extent would result from a four foot free field displacement beneath the dam; probably it would not be significant for only a one-foot displacement criterion.

4. An important point to study is the rate of flow that can be expected through a dam separation that might result from the hypothesized base displacement. What width crack opening would produce an unacceptable flood condition downstream? A major advantage of the concrete dam is that it will not erode away even if a joint separation passes a very significant flow of water, especially with the small reservoir stored by Seven Oaks Dam. In my opinion defensive measures are not needed, but an estimate should be made of the expected flow rate from a possible dam crack.

5. As noted above I think they probably are not necessary.

6. N/A

7. N/A

8. In my opinion, the only possible catastrophic failure mechanism for the roller compacted concrete dam would be a sliding failure of the concrete blocks on the base rock if the design criteria ensure adequate static overturning stability. Static sliding stability should be insured by neglecting any possible cohesion on the potential slicing surface and by use of a conservative coefficient of friction. Reasonable allowance must be made for uplift, but there is no need to consider an equivalent static earthquake force because the amount of possible sliding during the earthquake accelerations would not be catastrophic. A factor of safety of two would be more than adequate in such a static analysis, and the resistance to sliding could be increased if necessary by providing an arched plan form or by "keying" the concrete dam into the base work, or both.

9. It is my opinion that a satisfactory concrete gravity dam definitely can be built at this site, especially if the function of the dam is flood control. In addition, I believe that the extremely small probability that any fault break will occur beneath the base of the dam must be considered in deciding on the type of dam to be built at this location; the possibility that such fault displacement could be as great as 4 feet is considerably more remote, and does not need to be taken as credible in my opinion.

In conclusion I must point out that my comments are based on purely subjective opinion and cannot be supported by analysis, any more than can the postulated displacements at the base of the dam. However, it is important to consider carefully whether this proposed displacement design criterion is appropriate in this situation. Adopting excessively conservative design requirements is not good professional practice if they lead to unnecessary costs even if they are "safe". In my opinion it would be very desirable to appoint a Consultants Board to advise on such judgmental issues that affect the design of Seven Oaks Dam.

Ray W. Clough

QUESTIONS
FOR
SEISMICITY EXPERT
SEVEN OAKS DAM ALTERNATIVES

A. Provide the seismic parameters for design of a zoned earth-rockfi
embankment dam at the site including the following:

1. Fault

2. Distance to Dam Site

3. Activity Rate

   a. Recurrence Interval

4. Maximum Credible Earthquake

   a. Magnitude and Intensity at Epicenter

   b. Intensity and Maximum Ground Acceleration, Velocity, and Displacement at Site

5. Maximum Probable Earthquake

   a. Magnitude and Intensity at Epicenter

   b. Intensity and Maximum Ground Acceleration, Velocity, and Displacement at Site

6. Acceleration Time-History

7. Bracketed Duration at 0.05g

8. Predominant Period

B. Provide any differing seismic parameters from A for design of a rol
compacted concrete dam at this site.

C. Provide comments on the proposed design measures (joint spacing
separation at the foundation contact) addressing the postulated 4-f
displacement in the foundation of the roller compacted concrete dam.

Seismological Report

for

Seven Oaks Dam

for

Santa Ana River Basin, California

for

Geotechnical Branch, Los Angeles District

Corps of Engineers

by

Bruce A. Bolt
Registered Geologist and Geophysicist (California)

Contract DACW09-87-M-1734

March 30, 1987

ATTACHMENT

I. Introduction and Scope of Work

An assessment is now underway of alternatives for Upper Santa Ana River flood storage by the U.S. Army Corps of Engineers, Los Angeles District. An important part of this study is a quantitative estimation of geotechnical parameters required for the seismic evaluation of the foundation for either an earthfill embankment dam or a roller compacted concrete dam at the Seven Oaks Dam site.

This dam site is located approximately 2 km upstream from the mouth of Santa Ana River Canyon near the south branch of the San Andreas fault. According to the Phase I Assessment provided by the Corps of Engineers, the North Branch of the fault crosses the reservoir area about 1 km upstream from the proposed dam site. The rock wedge between the two branches of the San Andreas fault in this vicinity is composed of Pre-Cambrian gneiss and Cretaceous diorite. The rock is often highly sheared and fractured coves from exploratory holes indicate that the rock is fractured to depths of several hundred feet, with evidence from thin clay gouge slickenslides of movement by fault slip. At the dam site, the Santa Ana River has eroded a symmetric canyon into the fractured and weathered bedrock.

As requested, this report addresses the following seismological questions.

A. Provision of seismic parameters for design of a zoned earth-rock-filled embankment dam at the subject site including the following:

1. Properties of the seismic fault sources
2. Distances between seismic sources and the dam site

3. Earthquake activity rate

   a. Recurrence interval

4. Definition of the maximum credible earthquake

   a. Magnitude and seismic moment

   b. Intensity and maximum ground acceleration, velocity and displacement at the site

5. Definition of the maximum probable earthquake

   a. Magnitude and seismic moment

   b. Intensity characteristics such as maximum ground acceleration, velocity, and displacement at the site

6. Acceleration time history

7. Bracketed duration above 0.05g

8. Predominant period

B. Provision of any differing seismic parameters from Case A for design of roller compacted concrete dam at this site.

C. Provision of comments on the proposed design measures (joint spacing and separation at the foundation contact), addressing the postulated 4-foot displacement in the foundation and abutments of the roller compacted concrete dam.

II. Seismicity and Active Fault Information

The seismicity of this area near the San Bernardino Mountains (between the San Andreas, San Jacinto and Banning faults) has been summarized in the Phase I Assessment (D-14 to D-16) provided by the Corps of Engineers (31 December, 1986). Earthquake occurrence has also been addressed in an attachment by Clarence R. Allen (Letter 12/8/1984). Historically, and at present, the area has been the site of numerous small to moderate earthquakes (see Figure 7, Ziony, 1985) with the largest within 15 km of the site being a 1935 earthquake of magnitude 5.1, perhaps caused by slip near the Mill Creek fault. Although no great earthquakes have occurred in historical times from seismic sources near the San Bernardino Mountains, the geodetic , geological and seismological evidence together is quite convincing that earthquakes with magnitudes up to 8+ could be produced by elastic rebound of long sections of the major faults in the area.

Specifically, the site lies between the North Branch and the South Branch (San Bernardino segment) of the San Andreas fault. The Phase I Assessment quotes recent geological studies that it is likely that the presently active trace is the South Branch (about 2 km from the site). Professor Allen has discussed the evidence for Quaternary slip on the North Branch (Mill Creek fault), located about 1 km from the site. It is clear that the assessment of seismic intensity for dam design purposes would not differ for major fault slip on either Branch; for a magnitude 8+ earthquake the dam, site is in the near seismic field of either fault source.

III. Definition of Seismic Sources

As discussed in the last section, given the tectonic and seismic history of the area, the maximum credible earthquake for this site is one similar to the 1857 Fort Tejon earthquake on the southern San Andreas fault and the 1906 earthquake on the northern San Andreas fault. Such earthquakes are produced by long fault rupture (up to 430 km) with offsets of many meters, mainly in a right-lateral strike-slip sense. The magnitude of the 1906 earthquake was $M_s = 8.25$ as measured by seismographs recording in Europe.

A statistical estimate based on recorded seismicity for the recurrence time of the "maximum credible earthquake" is not feasible because of the short historical interval (going back approximately to 1800). A useful estimate can, however, be obtained from the newly developed geomorphological and stratigraphic methods of inferring average time between major earthquakes produced by slip on the San Andreas fault. The evidence acquired by G. Sieh (1978) in the Pallet Creek area north of this site indicates that there is a recurrence interval of 130 to 150 years between earthquakes large enough to cause liquefaction of sand and peat layers in that area. The main uncertainty is in the magnitude associated with each event and not all the earthquakes in the set treated by Sieh might reach magnitudes in excess of 8. Nevertheless, a value of 150 years is clearly appropriate as an interoccurrence time for an 8+ earthquake that would be produced by major slip of one or other branch of the San Andreas fault near the dam site. Because no such earthquake has occurred for almost 200 years it must be regarded as a likely event.

The estimation of a "maximum probable earthquake" is less straight-forward than that of the "maximum credible earthquake" because, as mentioned above, the time series of recorded earthquakes in the area is short. My approach to the problem is to consider a reasonable life expectancy for the dam structure and consider the greatest earthquake to likely occur within this time. A reasonable project operation interval is 50 years, say. As pointed out in the Phase I Assessment (p. D-64), various realistic published earthquake recurrence models predict that a 8+ magnitude event is "overdue" along the southern San Andreas fault. Thus, in terms of a 50 year time window, the maximum credible earthquake is also close in size to the maximum probable earthquake in this case. The seismic moment of such an earthquake is, from measurements in similar cases elsewhere, about 10^{28} dyne-cm.

The above argument is supported by work by U.S.G.S. in Southern California (Ziony, p. 80, 1985). The published conclusion is "the probability of occurrence of 1857-type earthquakes along the (southern San Andreas) fault is currently estimated to be about 1 percent per year but about 40 percent within the next 30 years. Thus an earthquake of $M_s = 8$ appears credible for ordinary planning and design purposes".

It is not necessary, however, for the main release of seismic energy from the fault dislocation to be produced at the nearest part of the San Andreas fault to the site. A more probable distance is 20 km (see Table 1).

IV. Estimation of Ground Motion Parameters

A. Zoned Earth-Rockfill Embankment Dam

In the previous section, the maximum credible earthquake was defined as a magnitude 8+ event produced by extensive right-lateral rupture of the nearby San Andreas fault. Surface fault ruptures involving elastic rebound or fault fling could occur within 2 km of the dam site in such a major dislocation. By extrapolation from similar major earthquakes such as the 1906 San Francisco earthquake, the major rupture may extend for over 200 km with horizontal surface offsets of up to 5 meters along the main fault trace. Given this seismic source, estimation of the strong ground intensity at the Seven Oaks dam site must proceed by extrapolation using seismological theory and ground motion measurements obtained by accelerographs in similar source and site situations. The construction is not direct because no records of strong motion have yet been obtained near a shallow right strike-slip faulting producing an earthquake with a magnitude $M_s > 7.5$.

The estimation method followed here to compute the requisite seismic intensity parameters and an appropriate synthetic strong motion accelerogram is as follows. The resulting values are listed in Table 1. First, the peak ground acceleration velocity and displacement are selected from the most appropriate recently published attenuation curves (e.g. Joyner and Boore, 1985, Seed and Idriss, 1985, Abrahamson and Bolt, 1982). The magnitude dependence of expected peak ground parameters very near to the fault source is not well known because of lack of large-earthquake data. Nevertheless, the limited elastic strength and rigidity of the strained rocks place limitations on the values of the supremum amplitudes (i.e. peak maxima of the waves). In addition, the depth of the slip

TABLE 1

Estimated (Mean) Ground Motion Parameters on Rock at Site

	Max. Credible Earthquake (Adjacent San Andreas fault)	Max. Probable Earthquake (in 50 years)
Source Distance to Dam Site (km) (Surface Rupture)	2	20
Magnitude (M_s)	8+	7.5 - 8.0
Seismic Moment (dyne-cm)	10^{28}	10^{27}
Recurrence Rate	150 ± 30 years	1 percent chance per year
Peak Horizontal Acceleration	0.70g	0.5g
Peak Horizontal Velocity (cm/sec)	90-105	70-80
Peak Horizontal Displacement (cm)	50-80	40-70
Bracketed Duration at ~~0.5~~ 0.05g (sec)	40-50	35
Predominant Period in ground velocity (sec)	0.2-10.0	0.3-8.0

surface where most seismic energy is released (below about 5 km in the San Andreas case) and the high wave attenuation in the fractured fault zone place a bound on the local peak accelerations. Consultation of empirical attenuation curves which extrapolate to higher magnitude suggest that an expected peak ground acceleration of 0.7g is an appropriate estimate in the present case. It is emphasized that the value is a mean value at a rock site. The effect of weathering and widespread cracking of the rock would be to reduce this estimate slightly. Similar correlations give peak horizontal velocities of the ground in the range of 90-105 cm/sec and peak ground displacements of 50-80 cm. Empirical curves (Bolt, 1973) for the bracketed duration of shaking above an acceleration of 0.05g yield an interval of 40-50 secs. For a magnitude 8+ earthquake there will probably be a persistent train (or "coda") of long-period (greater than 5 sec) surface waves with accelerations below 0.05g.

The predominant period window of the ground motion given in Table 1 is estimated from sets of amplitude spectra for strong ground motions available in California Strong Motion Instrumentation Program analyses and other literature (e.g. Seed and Idriss, 1982). Both Fourier spectra and response spectra (with small damping) indicate that for the largest earthquakes ($M_s > 7.0$) in the near field, maximum velocity ground response is in the range 0.2 to 10.0 sec (5 to 0.1 Hz) for (weathered) rock sites. At longer periods, near to a major fault rupture some enhancement of ground motions is expected (see discussion Section V).

Finally, parameters are also given in Table 1 for the dam site in the case of a maximum probable earthquake as defined in an earlier section.

The estimates have been obtained in the same way as for the maximum credible earthquake. There are only small differences in values, probably within the uncertainty (as measured by a reasonable standard deviation) for these parameters. The differences arise from adopting a greater distance for the source-to-site distance and a smaller seismic moment.

B. Roller Compacted Concrete Dam

The scope of work specifies that consideration be given to any differences in ground motion that should be considered for construction of a dam of the above type compared with construction of an earth rock-fill dam.

The strong ground shaking parameters estimated for the latter in IV-A above (see Table 1) were derived using field measurements from strong motion accelerometers for a rock site at an appropriate source-site distance. The values are independent of the type of engineering structure being considered. In particular, occurrence rate and size for both the maximum credible and maximum probable earthquakes are the same for the two types of dam being contemplated for Seven Oaks Dam.

Similarly, the ground motion intensity parameters were estimated from regressions of observed values, conditioned on appropriate ranges. In applications to design considerations, however, differences might arise due to different methods of structural analysis. The frequency range of the input motions must be appropriately broad to accommodate the full modal response of both types of dam. The peak parameter values given in Table 1 apply to waves in the range 5-10 Hz which is normal for scaling time histories and response spectra. Similarly, the bracketed duration applies to a spectral window with an upper bound of about 10 Hz.

Attention must also be given to the frequency range of uniform spectral power (wave amplitude squared) in time histories adopted for design purposes. This spectral uniformity is provided in the synthetic accelerogram provided in the next section. With this proviso, there appears to be no reason to modify the values in Table 1 for application to a roller-compacted concrete dam.

V. Synthetic Ground Acceleration Time-History

Because no field recordings in the near field of a M_s = 8+ strike-slip shallow-focus earthquake exist, a synthetic accelerogram must be constructed which is consistent with seismological theory of the appropriate source mechanism and propagation path. Kinematic scaling is carried out on ground acceleration, velocity and displacement and spectral content by extrapolation from a selected set of smaller earthquake recordings.

Several seismological aspects of the ground motion can be set down as criteria. First, the earliest part of the shaking consists mainly of P and back-scattered S waves. The requirement that the source produce the postulated maximum credible earthquake at the site suggests that the interval between the S and P onsets be from 3 to 5 secs (i.e. the focus no more than 25 km away). Secondly, because the site is close to the rebounding fault, the ground motion should contain a pulse at about the time of the major S wave arrival that models the fling of the fault as the rupture goes by the site. This longer period fling is of importance in a number of engineering contexts. A displacement SH pulse with periods of several seconds will propagate outwards and although not necessarily having the largest acceleration in the wave train, it may be associated with the greatest kinetic energy. For example, from studies of the damaged Olive View Hospital in the 1971 San Fernando earthquake (Bertero et al., 1978), failures in that structure apparently occurred during such a long duration pulse. It is now regarded as good practice to include in the appropriate portion of the near source record such longer period pulses to ensure that the longer period responses of a structure are realistic.

Further, in developing a synthetic time-history appropriate for this near-field site, there must be realistic constraints on the spectral components of the motion. This problem is treated by comparing pseudo-relative velocity spectra from seismic ground motions previously observed with that calculated for the synthetic record. The seismological proviso is however, that the synthetic record should have a response spectrum with somewhat higher spectral amplitudes in the long period range because of the longer fault slips and rise times associated with large magnitude earthquakes.

Few records which meet the above criteria are available in the published literature. The time history recommended as appropriate for the maximum credible earthquake is shown in Figure 1. It was numerically constructed by the method of Fourier decomposition (see Bolt, 1981), first for a major strike-slip earthquake on the Hayward fault, and later substantially modified (in conjunction with Dr. J. Valera of Earth Sciences Associates for a study by the City of San Francisco) for the near field a magnitude 8+ earthquake produced by rupture of the San Andreas fault in central California.

Figure 1 shows the horizontal component seismograms for acceleration, velocity and acceleration as scaled to satisfy the site-specific parameters of Table 1. The S minus P interval is appropriate for rupture commencing near to the dam site and a pulse-like displacement is incorporated in the first S wave packet. Longer period surface waves are contained in the coda and the most important 62 sec of shaking are provided.

The recommended synthetic time history has been checked to satisfy the spectral requirements mentioned above. First, the Fourier velocity

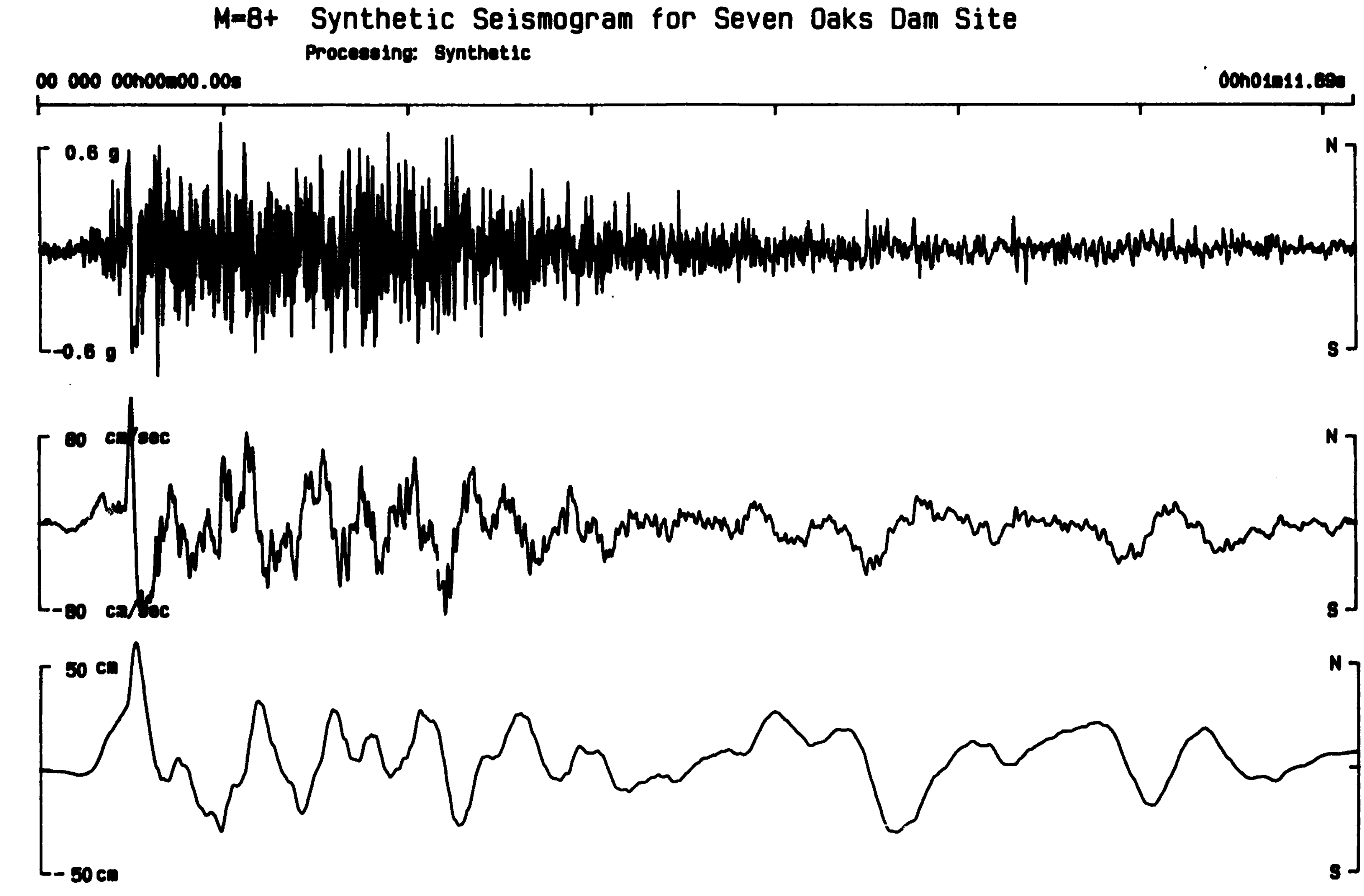
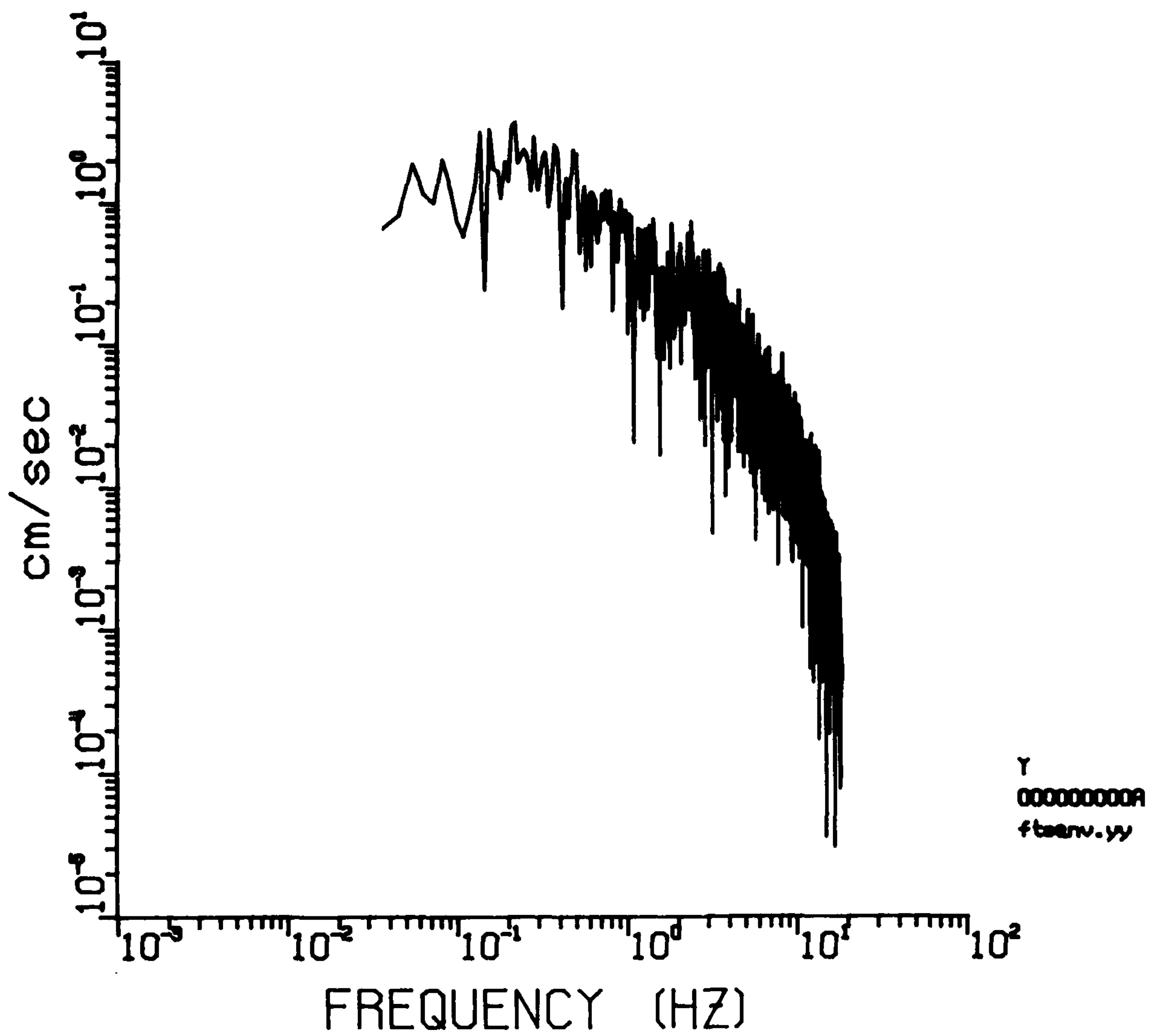


FIGURE 1 Time-histories: ground acceleration, velocity and displacement.

spectrum shown in Figure 2 shows a shape characteristic of many ground motion recorded in the near field of moderate earthquakes. There are no significant fluctuations or peaks or troughs in the velocity amplitude spectrum. Secondly, the normalized response spectrum (5 percent damping) for the recommended time history falls near the upper envelope of spectral shapes for similar curves from accelerograms at Taft, Koyna Dam and San Fernando (rock sites) and Imperial Valley (deep alluvium) - all smaller magnitude earthquakes. It is comparable with the spectral shape of the synthetic seismogram called Cal. Tech. Al.

The digital data for the accelerogram in Figure 1 are provided on a floppy disk. The peak digital value on the disk is normalized to unity (0.997) and requires scaling to 0.70 for input application in this case.

FIGURE 2 Undamped Fourier velocity spectrum for M=8+ synthetic seismogram.



VI. Effect of Nearby Significant Displacements

I am in general agreement with the definition and analysis of the probable occurrence and effects of significant fault displacements near to the dam, as set out in the Phase I Assessment (1986). In his consideration of the range of displacements that are credible on various components of the San Andreas fault system in the vicinity of the Seven Oaks Dam site, Professor Clarence Allen (Letter 12/8/1984) recommends the adoption of the assumptions that "as much as 4 feet of surficial fault displacement in any direction could take place, arbitrarily beneath the facility". The estimation of such displacements comes from field observations after previous large earthquakes associated with surface fault slip and from geomorphological and stratigraphic measurements along active faults (e.g. Bonilla, 1970; Sieh, 1978). A geotechnical case history of relevance in the present consideration is that of the proposed Auburn Dam the Sierra Nevada foothills in Northern California. Several geotechnical groups undertook extensive field work and evaluations of possible fault slip (e.g Idriss et al, 1977) that might occur near the site of the Auburn Dam. I was involved in the review of these studies.

My own evaluation would be that in the absence of any major active faults under the dam foundation or abutments it is perhaps more appropriate to consider the strain effects at the dam of 4 feet of displacement (or more) located on the nearby San Andreas fault planes. These effects would most likely be differential deformations and slips throughout the crustal volume adjacent to the fault, including shears along zones of weakness at the site.

Displacements would be carried from depth in the brittle rock up through the weaker (weathered) surficial materials. Variations in the strength and continuity of surface rock formations causing the strain dispersal by the shearing and cracking of rocks and finite slips over many joints and planes are, of course, unknown in detail so that prediction must be on the conservative side.

It might be expected, as pointed out in the Phase I Assessment (p. 8), that both horizontal and vertical displacements of the rock foundations under the dam could be accommodated by a multitude of adjustments within an earth-rock fill dam because of its low continuous rigidity and ability to deform differentially.

The measures for the design of a roller compacted concrete dam outlined in the Phase I Assessment (p. 19-21) to mitigate the effects of local elastic strain release indicate that engineering response to the problem is not completely infeasible. Because my own training is in applied mechanics and seismology rather than engineering I confine my response for comments on this design question largely to the general discussion above. With respect, however, to the models of deformation shown in Plate 6 of the Phase I Assessment, I stress again that, for the Seven Oaks damsite, the hazard is not likely to be a single major fault slip under the dam, as shown in the "worst case" sketches, but rather tensions and compressions due to the integrated displacements across the structure. Thus the close spacing of transverse vertical joints across the dam axis, designed to accommodate up to 4 feet of movement in any direction, overall might provide adequate response. The detail of faulting in the exposed foundation rock after the excavation would be the crucial factor in further evaluation of this question.

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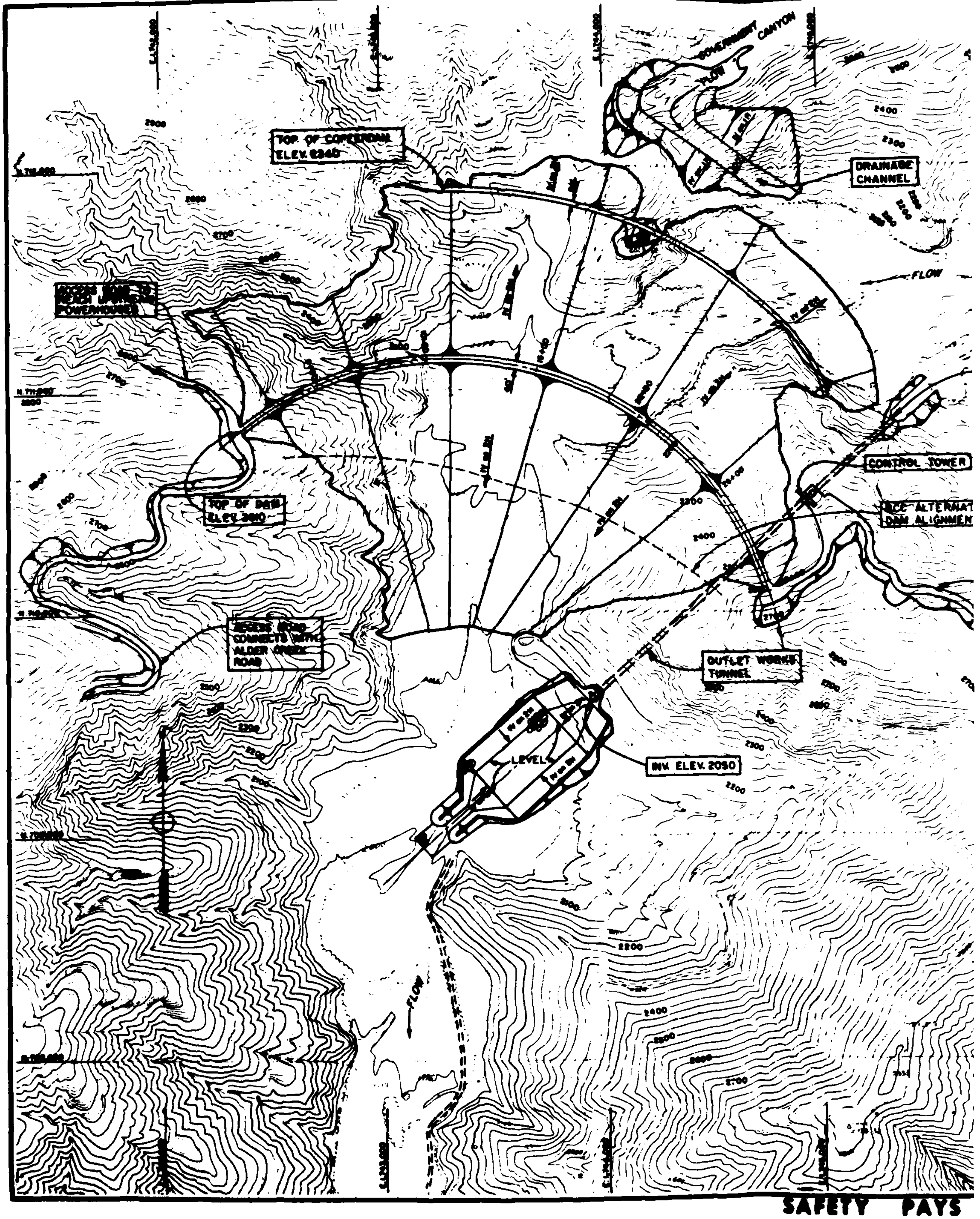
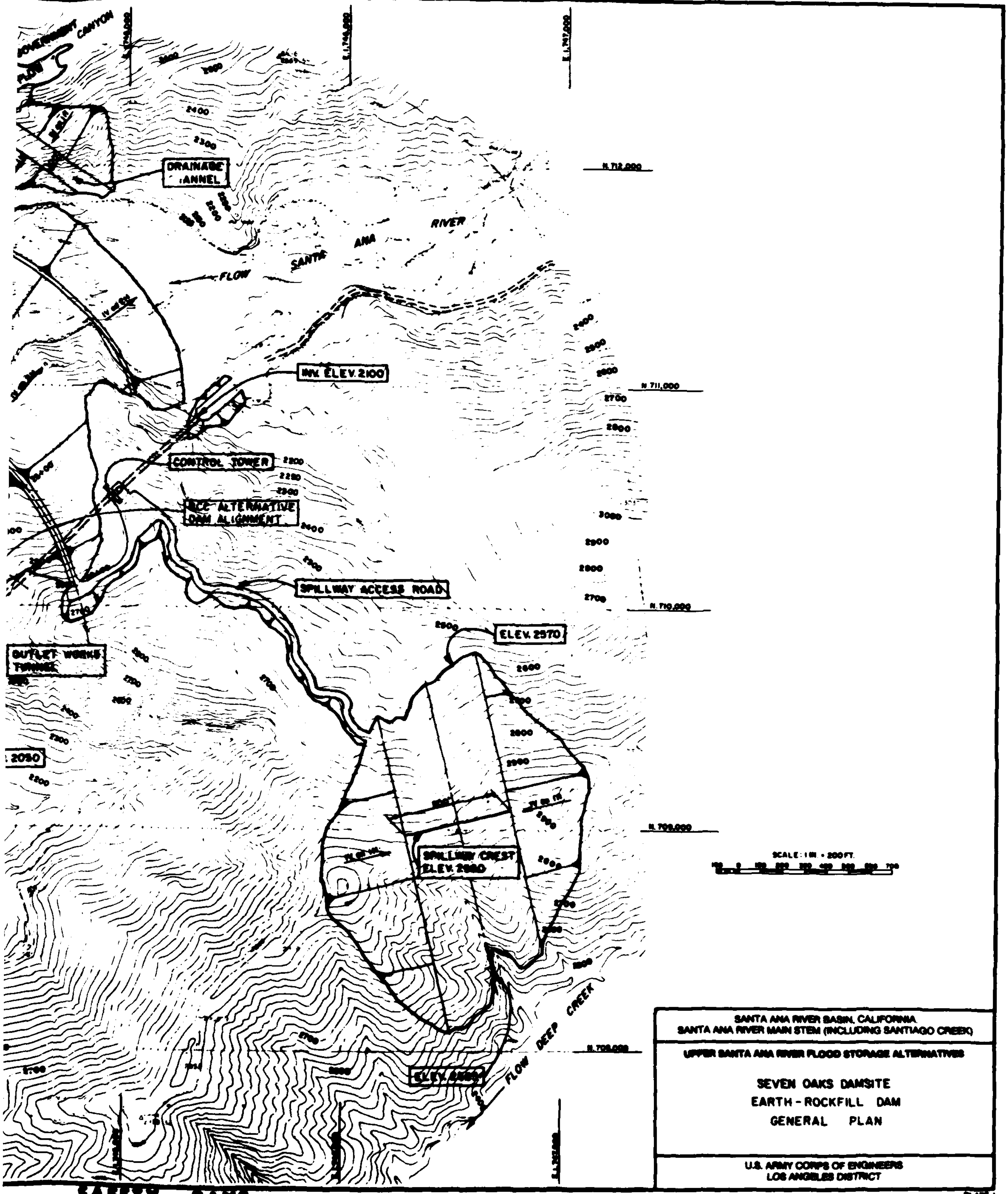
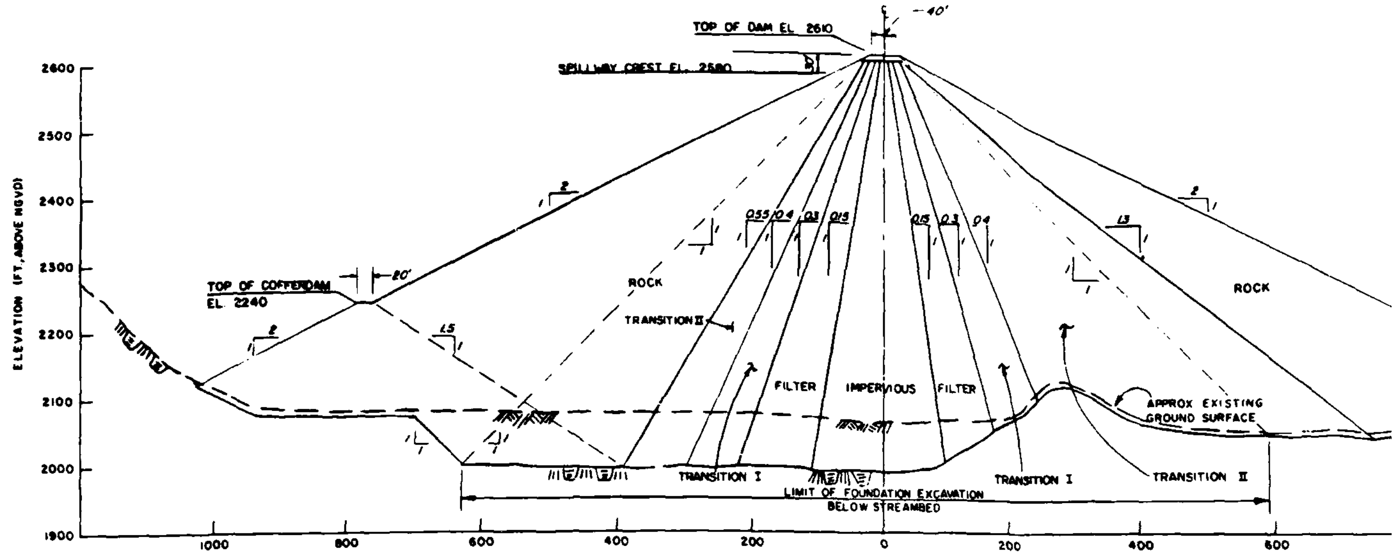
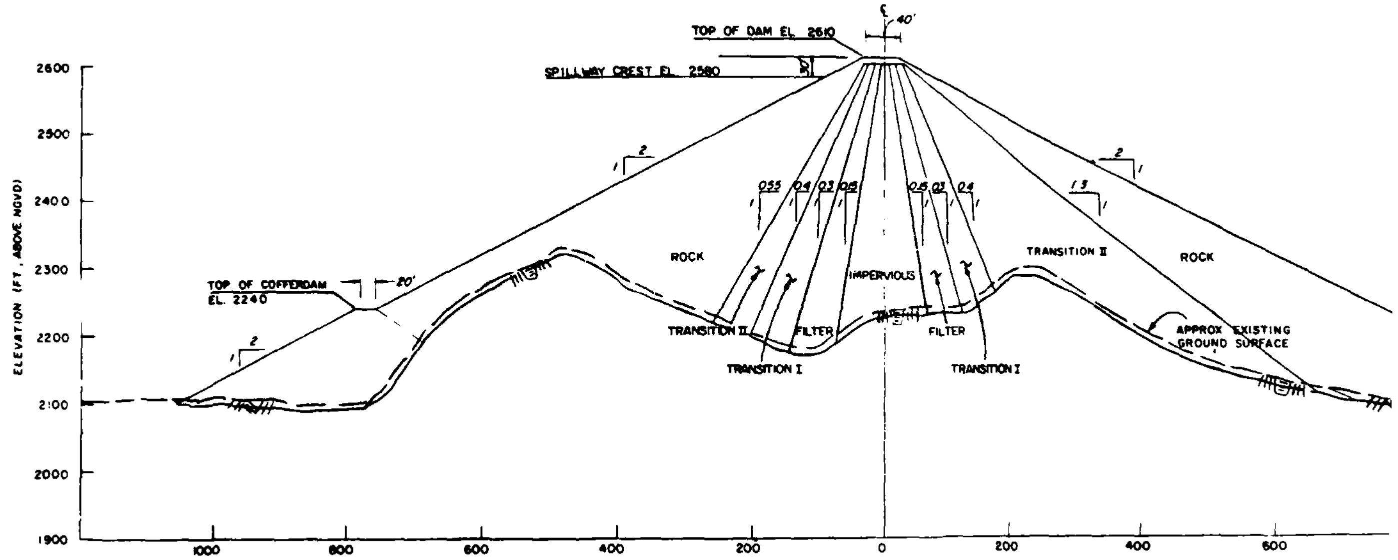

TOP OF COFFERDAM
ELEV. 2240
GOVERNMENT CANYON
DRAINAGE
CHANNEL
FLOW
CONTROL TOWER
OUTLET WORKS
TUNNEL
INV. ELEV. 2050
LEVEL



CANYON
DRAINAGE CHANNEL
RIVER
SANTA ANA
FLOW
INV. ELEV. 2100
CONTROL TOWER
RCC ALTERNATIVE DAM ALIGNMENT
SPILLWAY ACCESS ROAD
OUTLET WORKS TUNNEL
ELEV. 2570
SPILLWAY CREST ELEV. 2580
FLOW DEEP CREEK
N. 712,000
N. 711,000
N. 710,000
N. 709,000
N. 708,000
SCALE: 1 IN. = 200 FT.
SANTA ANA RIVER BASIN, CALIFORNIA
SANTA ANA RIVER MAIN STEM (INCLUDING SANTIAGO CREEK)
UPPER SANTA ANA RIVER FLOOD STORAGE ALTERNATIVES
SEVEN OAKS DAMSITE
EARTH-ROCKFILL DAM
GENERAL PLAN
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
PLATE



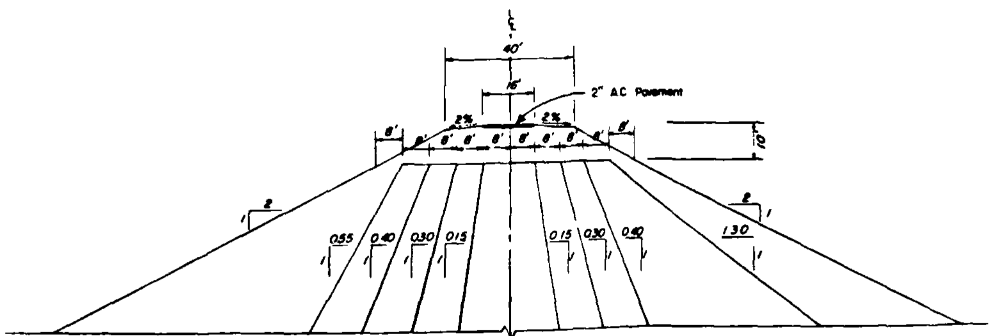
STA. 18+90



STA. 23+30

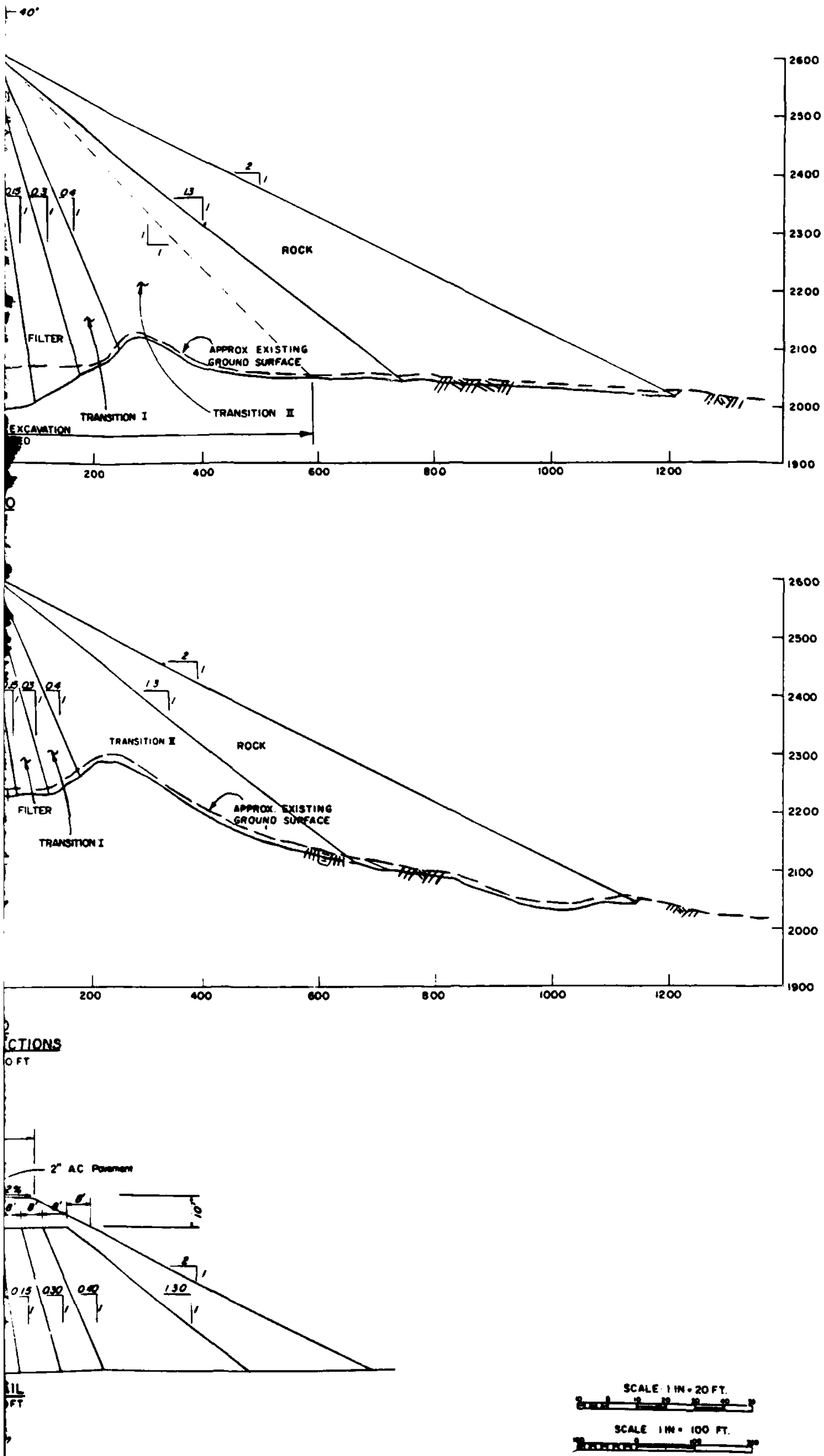
EMBANKMENT SECTIONS

SCALE: 1 IN = 100 FT



CROWN DETAIL

SCALE 1 IN = 20 FT



NOTES

1. EMBANKMENT SECTIONS ARE DRAWN LOOKING EAST.
2. BEDROCK SURFACE APPROXIMATED AND SMOOTHED.

SANTA ANA RIVER BASIN, CALIFORNIA
SANTA ANA RIVER MAIN STEM (INCLUDING SANTIAGO CREEK)

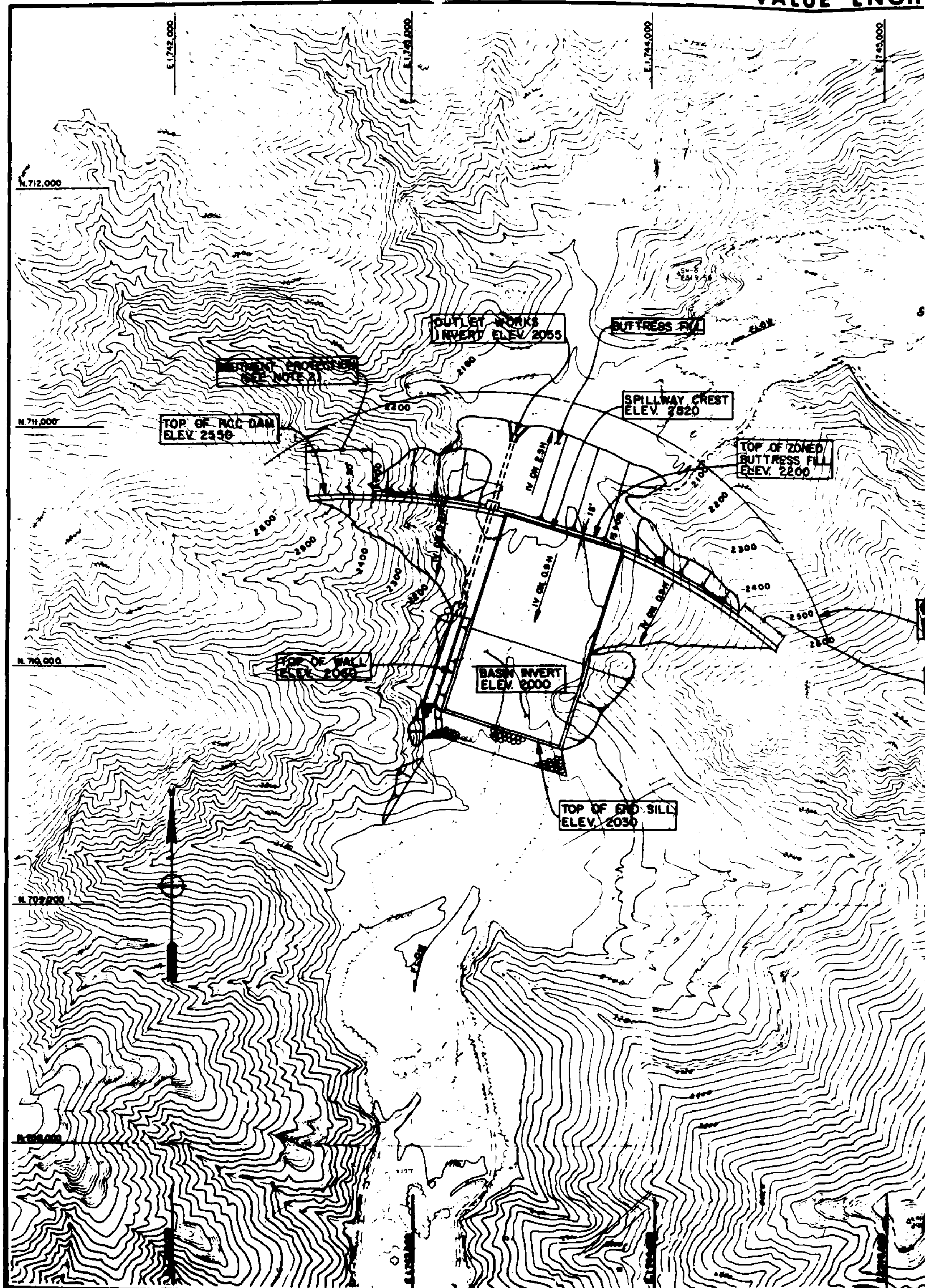
UPPER SANTA ANA RIVER FLOOD STORAGE ALTERNATIVES

SEVEN OAKS DAMSITE
EMBANKMENT CROSS SECTIONS

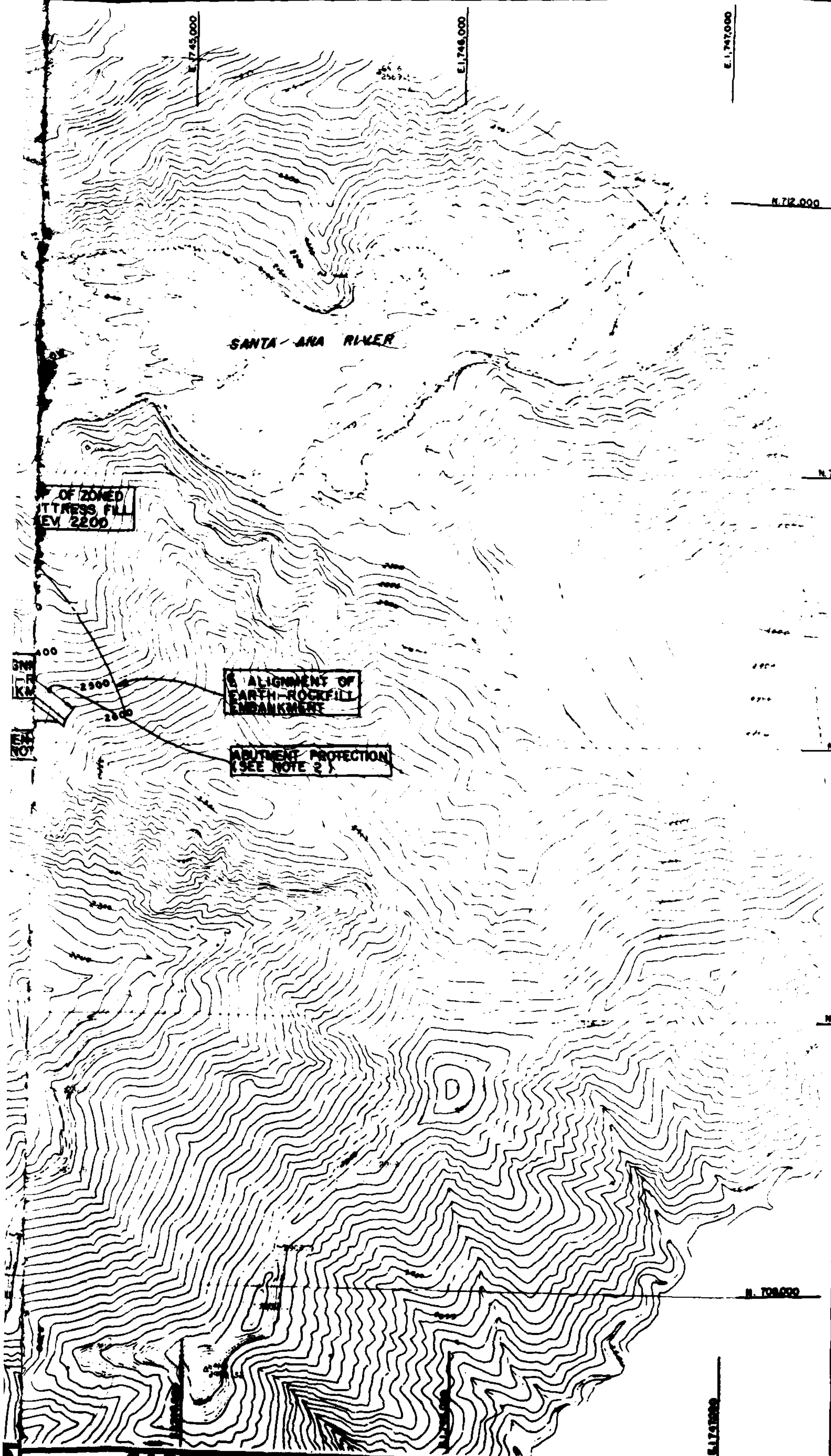
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 2

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING



VALUE ENGINEERING PAYS



NOTES:

1. RCC DAM DENOTES ROLLER-COMPACTED CONCRETE DAM.
2. ABUTMENT PROTECTION IS PROVIDED BY BACKFILLING THE UPSTREAM FOUNDATION TRENCH WITH ROCK II ZONE MATERIAL.
3. SEE PLATE 5 FOR BUTTRESS FILL AND ABUTMENT PROTECTION DETAILS.

SCALE: 1 IN. = 200 FT.

100 0 100 200 300 400 500 600 700

SANTA ANA RIVER BASIN, CALIFORNIA
SANTA ANA RIVER MAIN STEM (INCLUDING SANTIAGO CREEK)

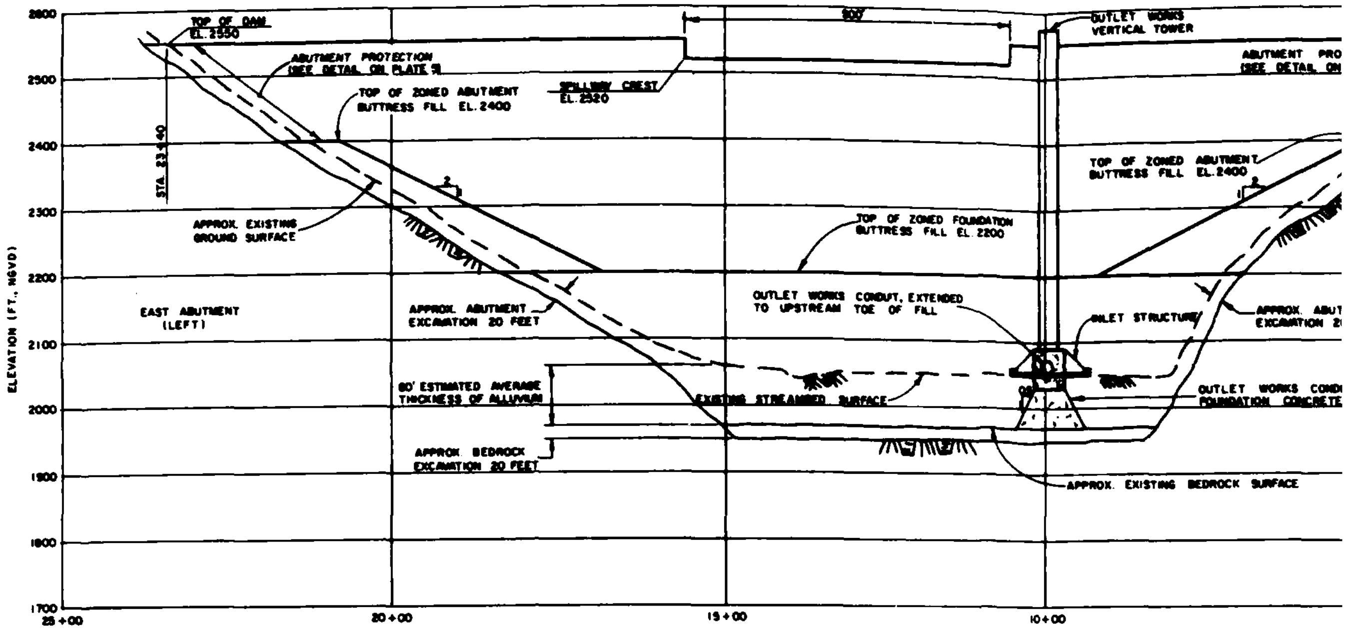
UPPER SANTA ANA RIVER FLOOD STORAGE ALTERNATIVES

SEVEN OAKS DAMSITE
RCC DAM
GENERAL PLAN

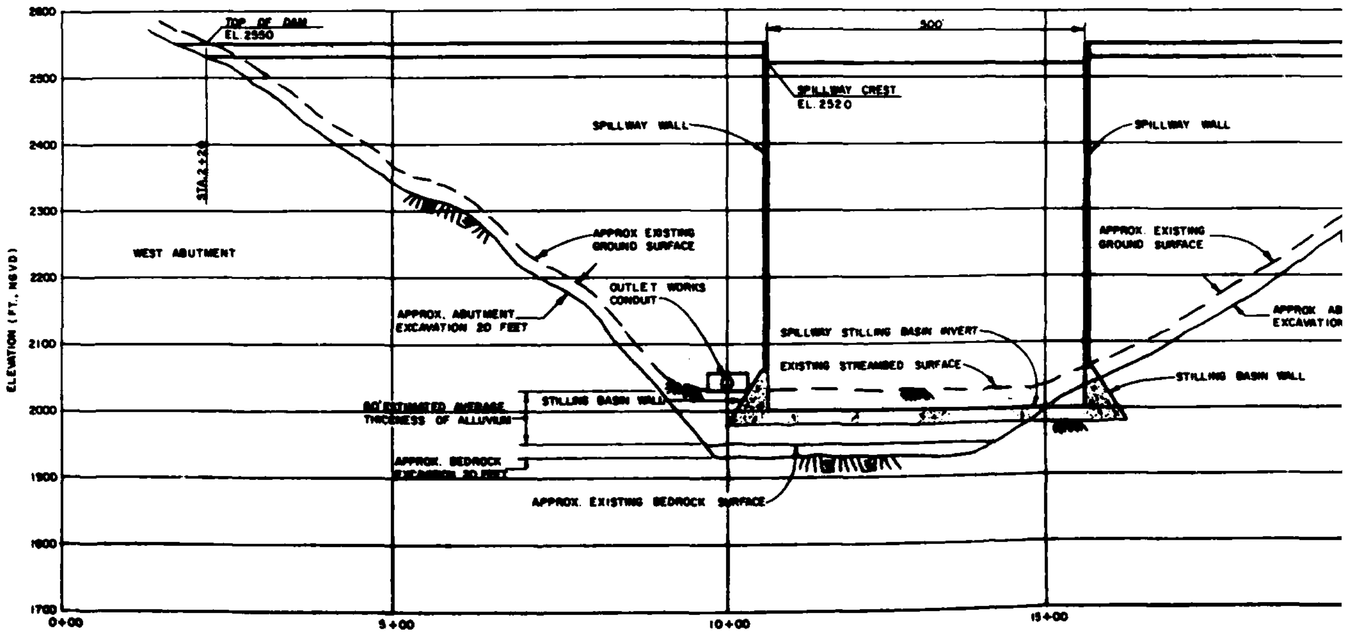
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

SAFETY PAYS

PLATE 3

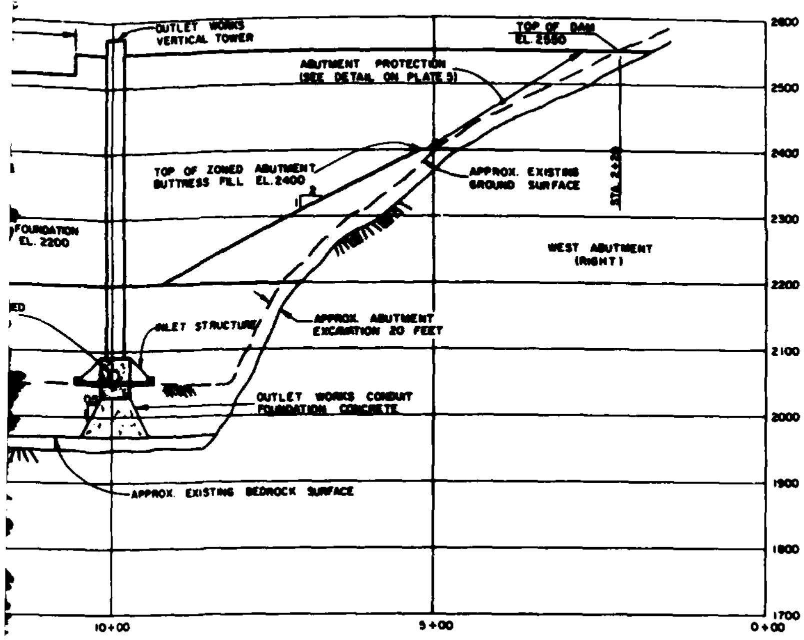


RCC DAM UPSTREAM ELEVATION
SCALE: 1 IN. = 100 FT.

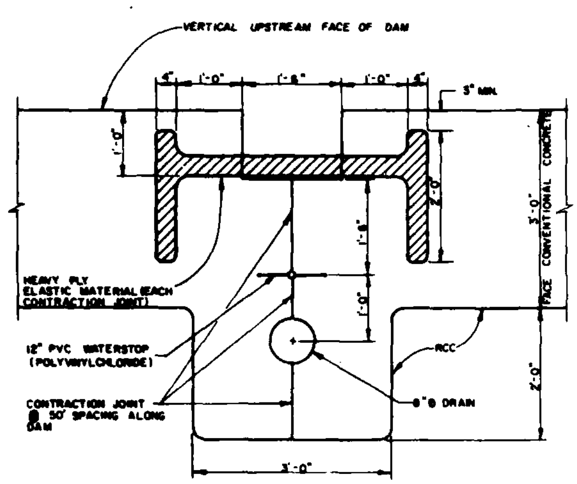


RCC DAM DOWNSTREAM ELEVATION
SCALE: 1 IN. = 100 FT.

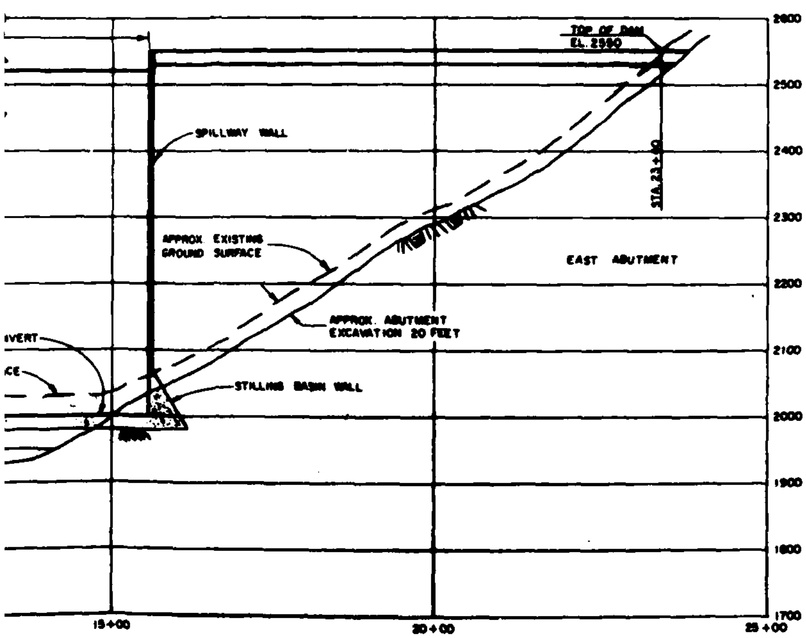
ENVIRONMENTAL
ENHANCEMENT
THRU ENGINEERING



LEVATION
FT.



VERTICAL CONTRACTION JOINT DETAIL
PLAN VIEW
SCALE: 1 IN. = 1 FT.



ELEVATION
T.

NOTE.
1. RCC DAM DENOTES ROLLER-COMPACTED CONCRETE DAM.

SCALE: 1 IN. = 1 FT.
SCALE: 1 IN. = 100 FT.

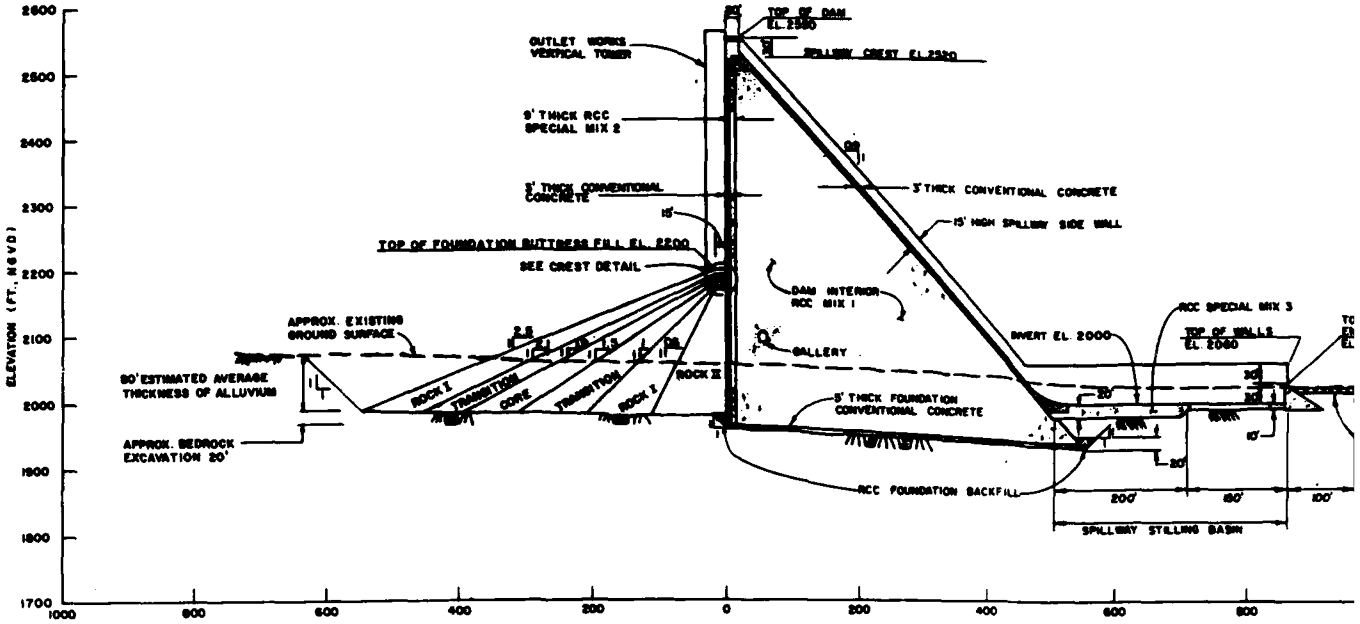
SANTA ANA RIVER BASIN, CALIFORNIA
SANTA ANA RIVER MAIN STEM (INCLUDING SANTIAGO CREEK)

UPPER SANTA ANA RIVER FLOOD STORAGE ALTERNATIVES

SEVEN OAKS DAMSITE
RCC DAM
PROFILES AND DETAIL

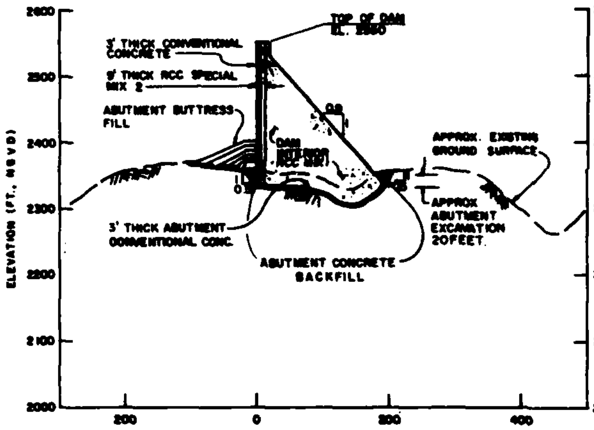
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 6



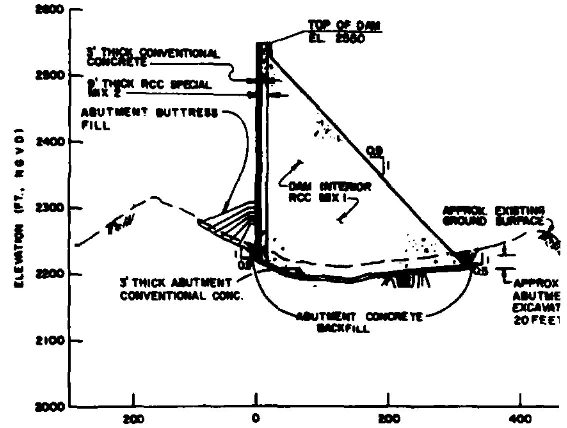
SECTION AT STREAMBED

SCALE: 1 IN.=100 FT.



SECTION AT RIGHT ABUTMENT

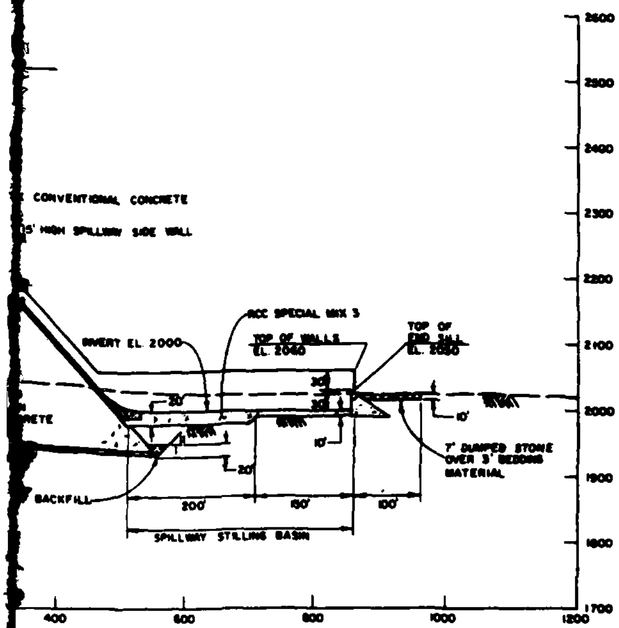
SCALE: 1 IN. = 100 FT.

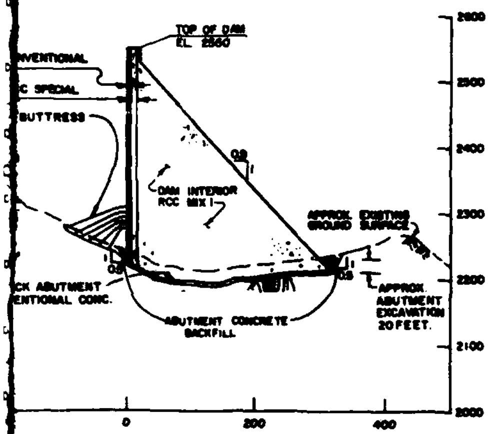


SECTION AT LEFT ABUTMENT

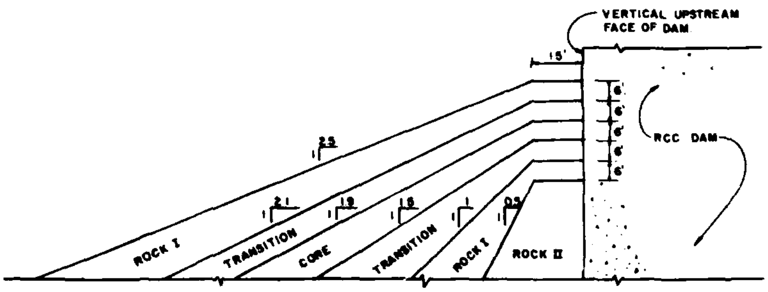
SCALE: 1 IN. = 100 FT.

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

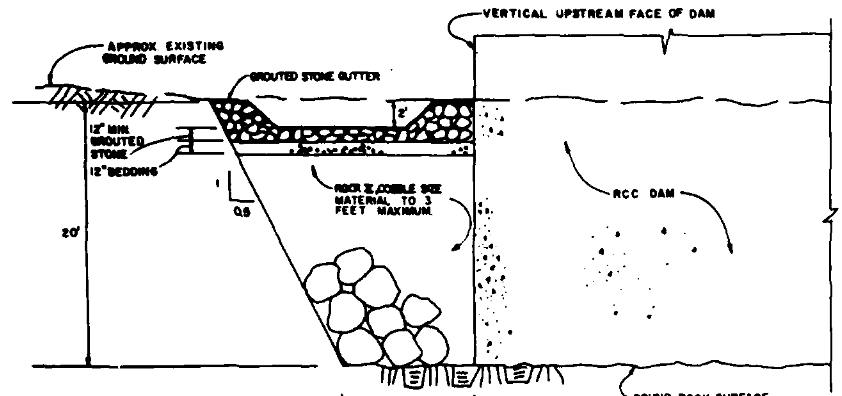




SECTION AT LEFT ABUTMENT
SCALE: 1 IN. = 100 FT.



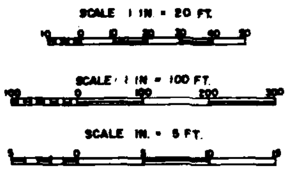
BUTTRESS FILL CREST DETAIL
SCALE: 1 IN. = 20 FT.



NOTE:
ABUTMENT CONCRETE BACKFILL IS NOT PROVIDED ABOVE ELEVATION 2400 FT.

ABUTMENT PROTECTION DETAIL (ABOVE ELEVATION 2400 FEET)
SCALE: 1 IN. = 5 FT.

NOTES:
1. ROCK I ZONE IS PERVIOUS MATERIAL TO 1 FOOT MAXIMUM.
2. ROCK II ZONE IS COBBLE SIZE MATERIAL TO 3 FEET MAXIMUM.
3. RCC DAM SECTIONS ARE DRAWN LOOKING EAST
4. RCC DAM DENOTES ROLLER-COMPACTED CONCRETE DAM



DATUM IS NATIONAL GEODETIC VERTICAL DATUM OF 1929

SANTA ANA RIVER BASIN, CALIFORNIA
SANTA ANA RIVER MAIN STEM (INCLUDING SANTIAGO CREEK)

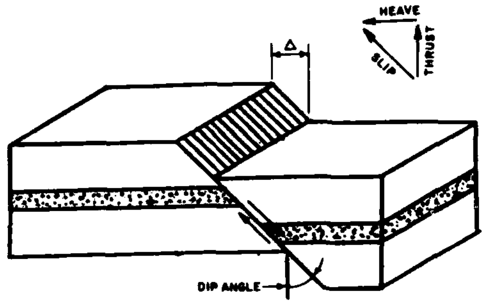
UPPER SANTA ANA RIVER FLOOD STORAGE ALTERNATIVES

SEVEN OAKS DAMSITE
RCC DAM
CROSS SECTIONS AND DETAILS

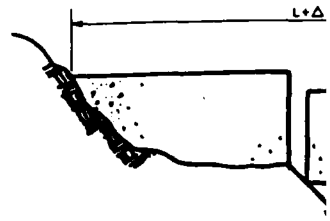
U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

PLATE 5

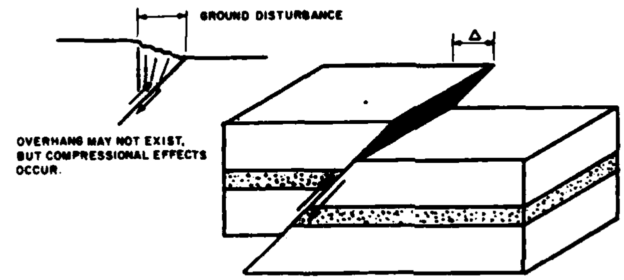
VALUE ENGINEERIN



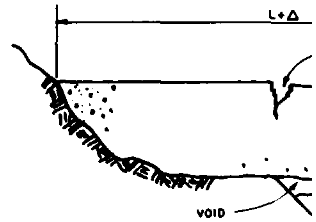
NORMAL FAULT MOVEMENT
NOT TO SCALE



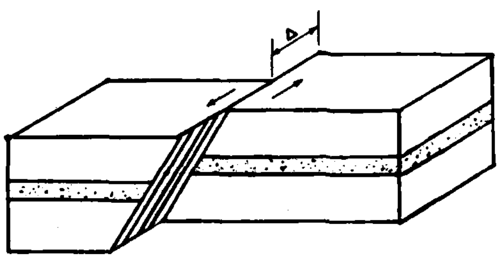
TENSION FAILURE
NO



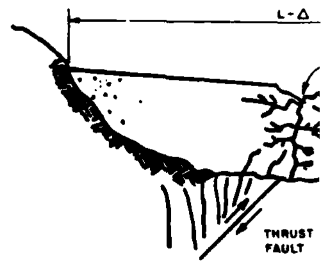
THRUST FAULT MOVEMENT
NOT TO SCALE



TENSION FAILURE
NOT TO S



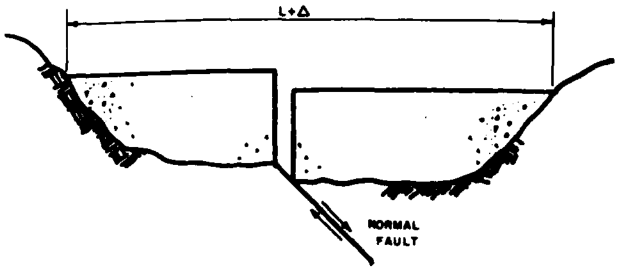
LATERAL FAULT MOVEMENT
NOT TO SCALE



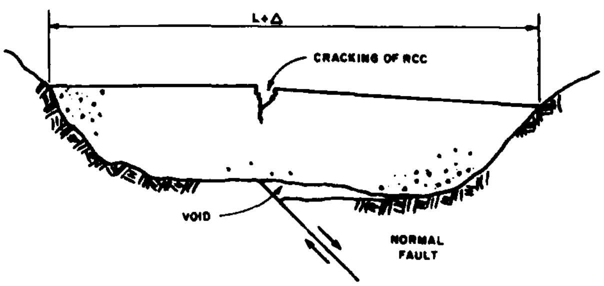
COMPRESSION
NOT TO SC

ENVIRONMENTAL ENHANCEMENT THRU ENGINEERING

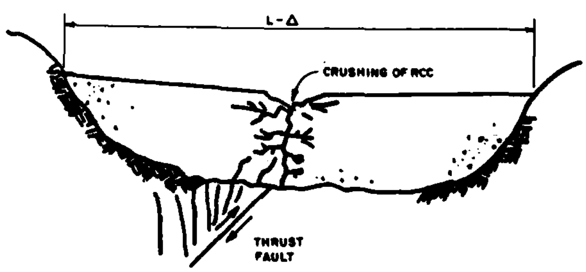
SAFETY PA



TENSION FAILURE (1) SEPARATION OF JOINTS
NOT TO SCALE



TENSION FAILURE (2) BRIDGING
NOT TO SCALE



COMPRESSION FAILURE
NOT TO SCALE

NOTES:

1. RCC DAM DENOTES ROLLER-COMPACTED CONCRETE DAM.
2. Δ DENOTES DISPLACEMENT.
3. L DENOTES LENGTH.

SANTA ANA RIVER BASIN, CALIFORNIA SANTA ANA RIVER MAIN STEM (INCLUDING SANTIAGO CREEK)
UPPER SANTA ANA RIVER FLOOD STORAGE ALTERNATIVES
SEVEN OAKS DAMSITE POSSIBLE FAULT MOVEMENTS AND RCC DAM FAILURES
U.S. ARMY CORPS OF ENGINEERS LOS ANGELES DISTRICT

SEDIMENT TRANSPORT STUDY

FINAL

SEVEN OAKS DAM
CHANNEL STABILIZATION DESIGN
AND RIVER SEDIMENT TRANSPORT STUDY

Submitted to

United States Army Corps of Engineers
Los Angeles District
300 North Los Angeles Street
Los Angeles, California 90053

Submitted by

Simons, Li & Associates, Inc.
3555 Stanford Road
P.O. Box 1816
Fort Collins, Colorado 80522

CA-COE-17

RDF276, 279/R1025

June 22, 1987

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I. INTRODUCTION

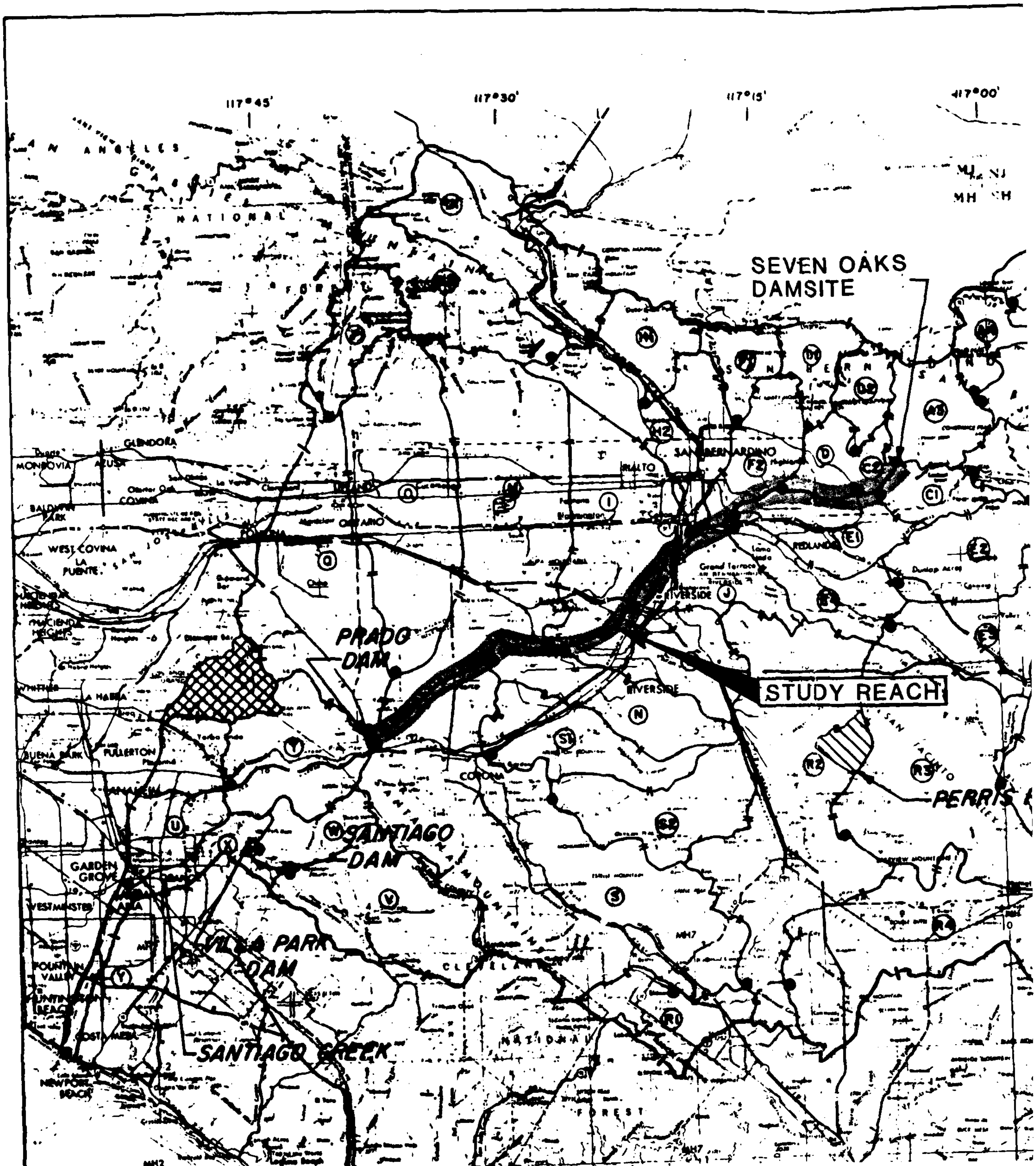
The purpose of this study is to analyze the impacts resulting from construction of the proposed Seven Oaks Dam, investigate various alternatives to mitigate the adverse impacts, and prepare a preliminary design of the selected alternative. Particular attention will be paid to impacts on the channel stability of the Santa Ana River between the dam site and the Greenspot Road bridge, and on aggradation and degradation trends in the river between Greenspot Road and Prado Dam. In conjunction with the channel stability analysis, the impact of the dam on all structures, pipelines, and other facilities between the dam site and Greenspot Road will be analyzed. The channel stability analysis upstream of Greenspot Road will involve a fairly rigorous quantitative level of detail, while the aggradation/degradation analysis downstream of Greenspot Road will be conducted at a less rigorous level using the sediment continuity principle.

The Seven Oaks Dam site is located on the Santa Ana River in San Bernardino County about 6.6 miles northeast of Redlands, California (see Figure 1.1). It is situated in the Upper Santa Ana Canyon approximately one mile upstream of the Greenspot Road bridge. Southern California Edison's Santa Ana River Powerhouse #3 is located approximately one-half mile downstream of the dam site. The study reach extends along the Santa Ana River from the proposed dam downstream to existing Prado Dam, a distance of approximately 40 miles.

The list of references provided at the end of this report includes several studies which are directly related to the current study. Also included are two studies conducted by Simons, Li & Associates, Inc. which were not related to the current study but did provide some useful information.

The current study required coordination with the Los Angeles District COE and the Portland District COE, through the Los Angeles District, concerning hydrology for the Santa Ana River, the proposed operating schedule for Seven Oaks Dam, and the location of proposed structures related to Seven Oaks Dam.

This study is being performed by SLA for the Los Angeles District, Corps of Engineers under contract number DACW09-86-D-0016.


117°45'
117°30'
117°15'
117°00'
SEVEN OAKS
DAMSITE
PRADO
DAM
STUDY REACH
SANTIAGO
DAM
VILLA PARK
DAM
SANTIAGO CREEK
PERRIS
SAN BERNARDINO
REDLANDS
RIVERSIDE
CORONA
FULLERTON
ANAHEIM
GARDEN
GROVE
WESTMINSTER
COSTA MESA
NEWPORT
BEACH
WEST COVINA
GLENDORA
AZUSA
MONROVIA
COVINA
BALDWIN
PARK
ONTARIO
RIALTO
BUENA PARK
LA HABRA
NATIONAL
FOREST

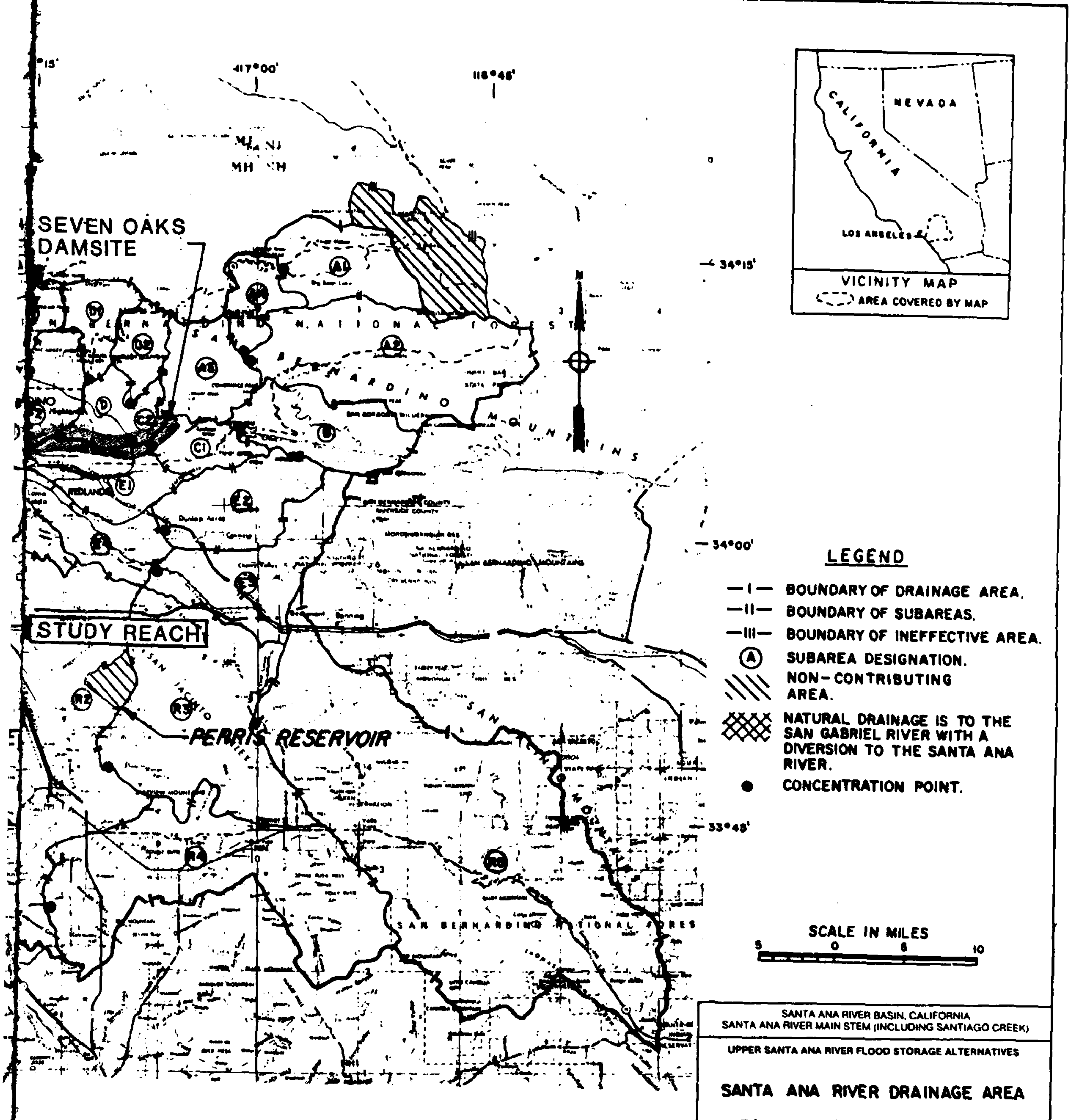


Figure 1.1. Location map.

II. DESCRIPTION OF STUDY CONDITIONS AND EXISTING DATA

2.1 Reach Description and Channel Geometry

Upstream of Greenspot Road, the Santa Ana River passes through the Upper Santa Ana Canyon. The canyon bottom width varies from approximately 500 feet near the dam site to nearly 1,500 feet at Greenspot Road. The average thalweg slope in this portion of the study reach is approximately 155 feet per mile (0.0294). The canyon walls slope at approximately 1 vertical to 1 horizontal. The channel contains a high percentage of boulders up to 6 feet in diameter and is interspersed with coarse sand and cobbles. Vegetation is sparse in the channel bottom and moderately dense on the canyon walls. Southern California Edison (SCE) is responsible for much of the development in the canyon. SCE operates three hydroelectric powerhouses in the Upper Santa Ana Canyon. Two powerhouses are located upstream of the Seven Oaks Dam site out of the study reach. The third powerhouse is located in the study reach, downstream of the dam site, approximately one-half mile upstream of Greenspot Road. In association with the third powerhouse, SCE also operates and maintains several auxiliary buildings along the east side of the canyon, and a bridge and flume which span the channel. These additional facilities are also located approximately one-half mile upstream of Greenspot Road. Other development in the canyon includes several pipelines and canals (administered by various water districts), a private residence, a cobble weir (at the SCE bridge), a flow diversion structure, and two USGS stream gaging stations. Also located in the Santa Ana Canyon is an area containing Eriastrum densifolium subspecies sanctorum (hereafter referred to as Eriastrum), a proposed endangered plant species. These structures, facilities, and other features are identified on Sheet 1.

Downstream of Greenspot Road, the Santa Ana River becomes relatively flat with the thalweg slope ranging from approximately 0.018 (95 feet per mile) near Greenspot Road to about 0.003 (15 feet per mile) near Prado Dam. Except at the Riverside levees, a fairly broad flood plain accompanies the low-flow channel. The bed material decreases in size from a large percentage of cobbles at Greenspot Road to medium and fine sands near Prado Dam. Vegetation in the channel is generally sparse from Greenspot Road to downstream of Interstate 10 (approximately 12 miles). Moderately dense vegetation continues downstream for about 16 miles where, upstream of Hamner Avenue, vegetation becomes very dense. Approximately 22 bridges span the channel along this por-

tion of the study reach. Development along the river is generally sparse to moderate with increased urbanization near the city of Riverside.

Bed profiles for the study reach are available from three sources. The COE profile was based on USGS quadrangle maps dated 1967. Flood insurance studies covering most of the study reach which were conducted between 1972 and 1978 provide a composite bed profile. SLA measured bed elevations from known bridge elevations in 1984 to define a current bed profile. Supplemental information for the SLA profile was provided by 1984 topography (COE) upstream of Greenspot Road. Only minor profile changes are indicated for the period from 1967 to 1984 except for a reach approximately 2 miles long, downstream of I-10, where 15 to 20 feet of degradation has occurred. It is believed this was caused by construction and dredging operations in the vicinity of I-10. Generally, the channel upstream of I-10 appears to be degradational, while the channel downstream of the Riverside levees is slightly aggradational. The three bed profiles are given in Table 2.1. The general aggradation/ degradation trends can be seen in Figure 2.1.

2.2 Summary of the Proposed Seven Oaks Dam Design

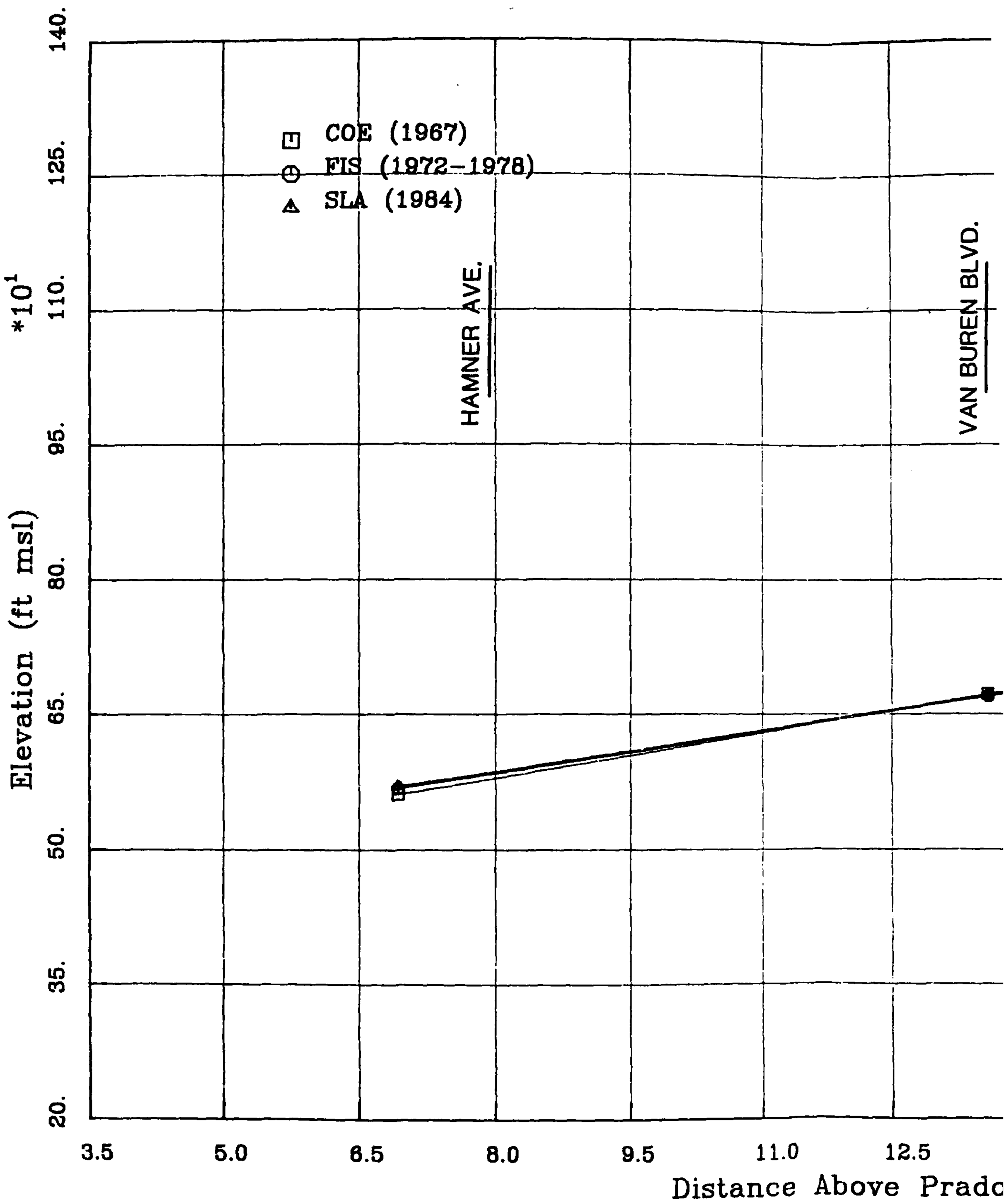
At present the dam design is not complete. This analysis assumes that the proposed Seven Oaks Dam consists of a curving earth-rock fill embankment about 3,000 feet long with a maximum height of 550 feet located at the Government Canyon/Santa Ana River confluence approximately one mile upstream of Greenspot Road (see Sheet 1). The dam would have an 18-foot wide, 17-foot high outlet tunnel about 1,800 feet long. The tunnel exit portal would discharge normal operation releases at a point above the existing channel bed on the east canyon wall. An ungated emergency spillway would be excavated in a rock saddle southeast of the dam embankment. Spillway flows would discharge into Deep Creek returning to the Santa Ana River between Greenspot Road and the SCE Powerhouse #3.

2.3 Bed-Sediment Data

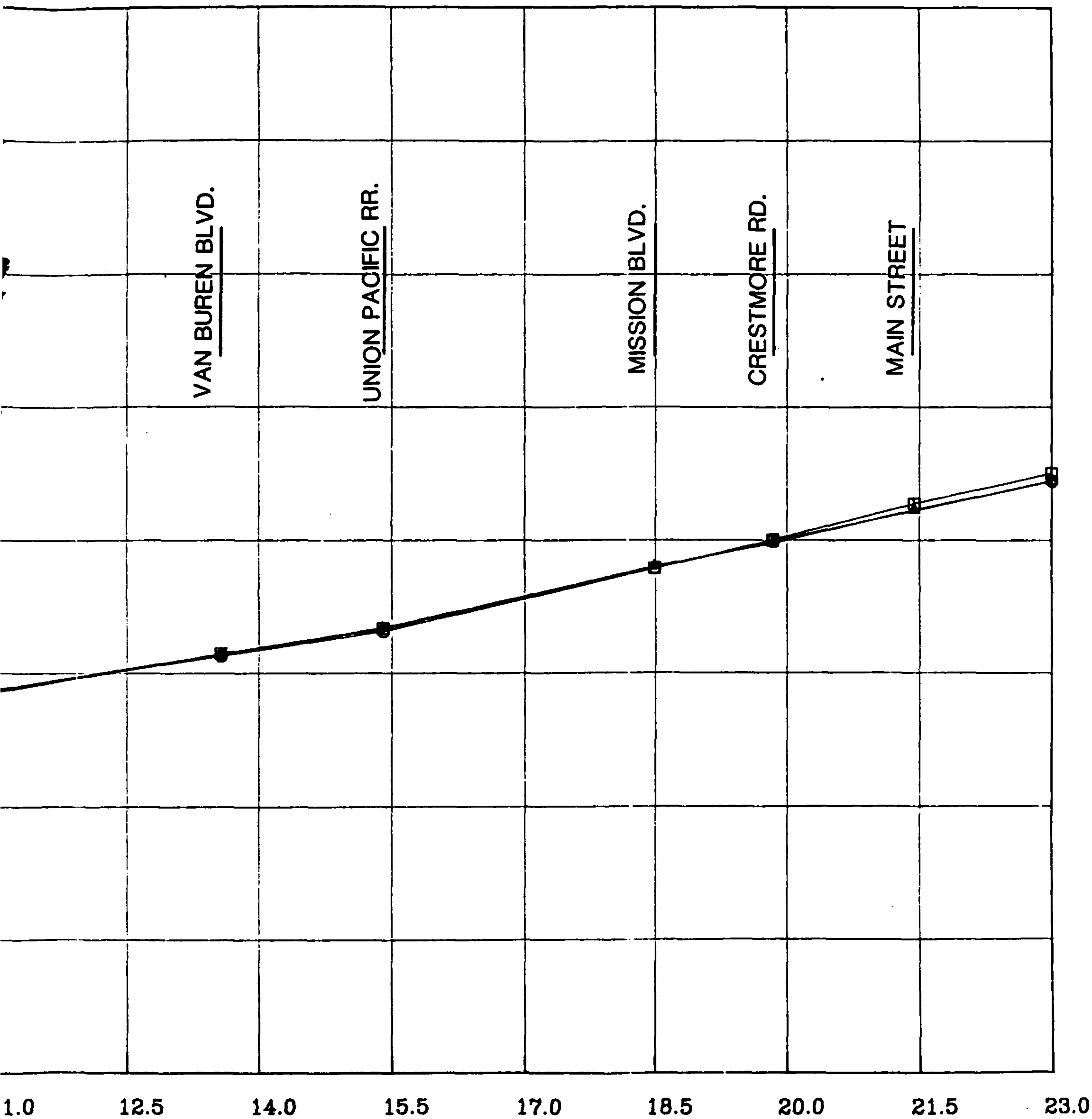
Representative bed-material gradation curves for the study reach were developed based on available data and samples collected specifically for this study. Gradation curves for three test trenches in the vicinity of the proposed dam embankment were reported by the COE (1985). However, the methods used in collecting these samples have since been questioned by the Los Angeles

Table 2.1. Santa Ana Riverbed Profiles.

Location	Distance From Prado Dam (River Miles)	Bed Elevation (ft) COE 1967	FIS 1972-1978	SLA 1984 & 1987
Hamner Ave.	6.92	561	568	570
Van Buren Blvd.	13.57	672	669	671
Union Pacific RR	15.40	700	697	701
Mission Blvd.	18.49	768	768	770
Crestmore Rd.	19.84	800	799	797
Main St.	21.44	841	--	834
8th St.	24.23	902	893	892
S.P. Railway	24.54	908	902	899
Mt. Vernon Ave.	25.74	941	926	927
S.P. Railway	26.11	950	928	930
Interstate 10	26.38	957	941	942
E St.	26.88	970	960	966
Waterman Ave.	27.80	995	989	993
AT & SF Railway	28.27	1,010	1,008	1,008
Tippecanoe Ave.	29.10	1,043	1,040	1,035
Alabama Rd.	32.15	1,188	1,180	1,180
Orange St.	33.74	1,288	--	1,286
Greenspot Rd.	38.36	1,850	--	1,848
SCE Bridge	39.03	--	--	1,949
Proposed Outlet Works	39.48	--	--	2,019

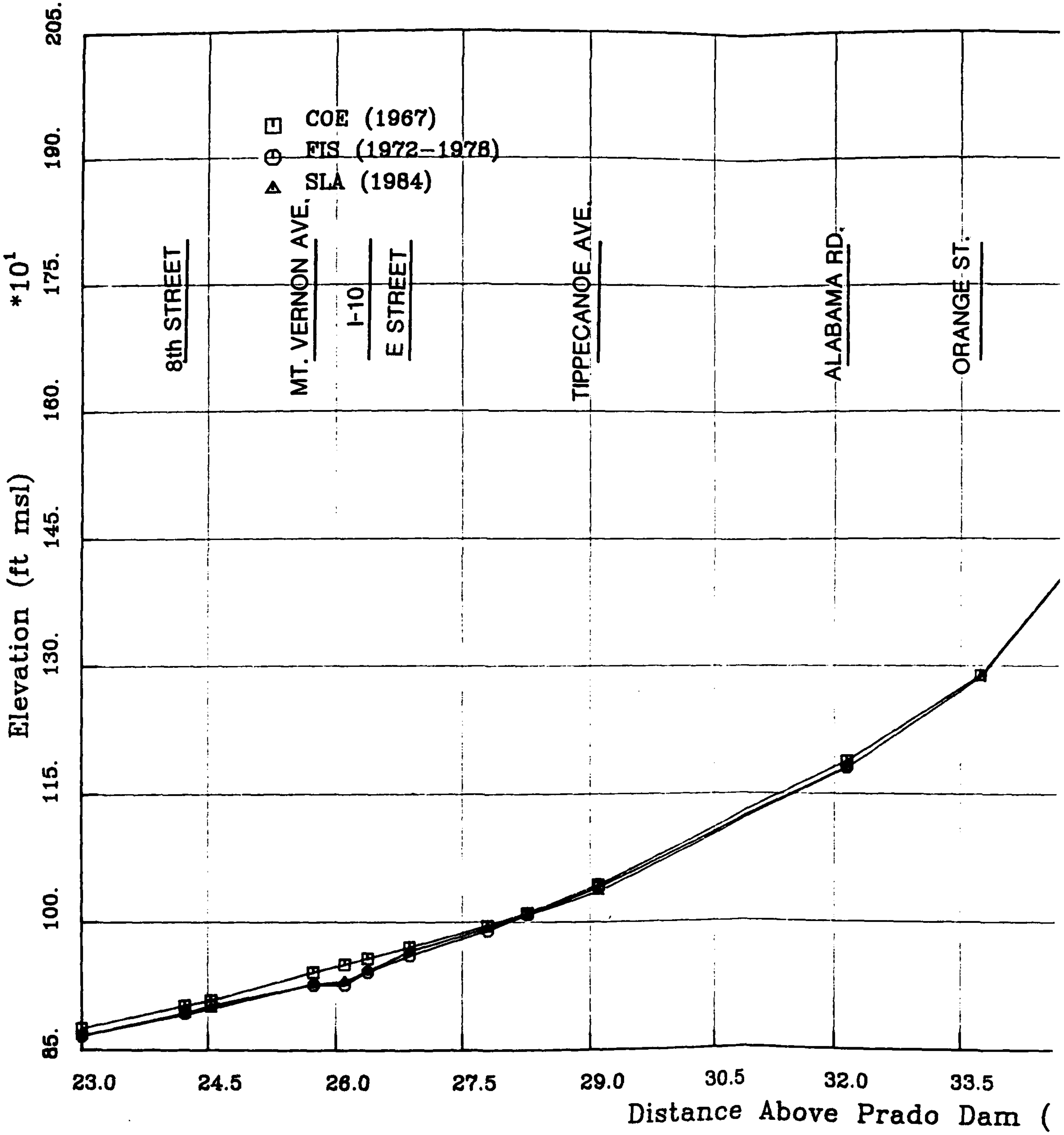


COE (1967)
FIS (1972-1978)
SLA (1984)
HAMNER AVE.
VAN BUREN BLVD.
Elevation (ft msl)
*10^1
140.
125.
110.
95.
80.
65.
50.
35.
20.
3.5
5.0
6.5
8.0
9.5
11.0
12.5
Distance Above Prado



ce Above Prado Dam (miles)

Figure 2.1. Historic bed profiles for the Santa Ana River.



COE (1967)
FIS (1972-1978)
SLA (1984)
8th STREET
MT. VERNON AVE.
I-10
E STREET
TIPPECANOE AVE.
ALABAMA RD.
ORANGE ST.
Elevation (ft msl)
*10[1]
85.
100.
115.
130.
145.
160.
175.
190.
205.
23.0
24.5
26.0
27.5
29.0
30.5
32.0
33.5
Distance Above Prado Dam (

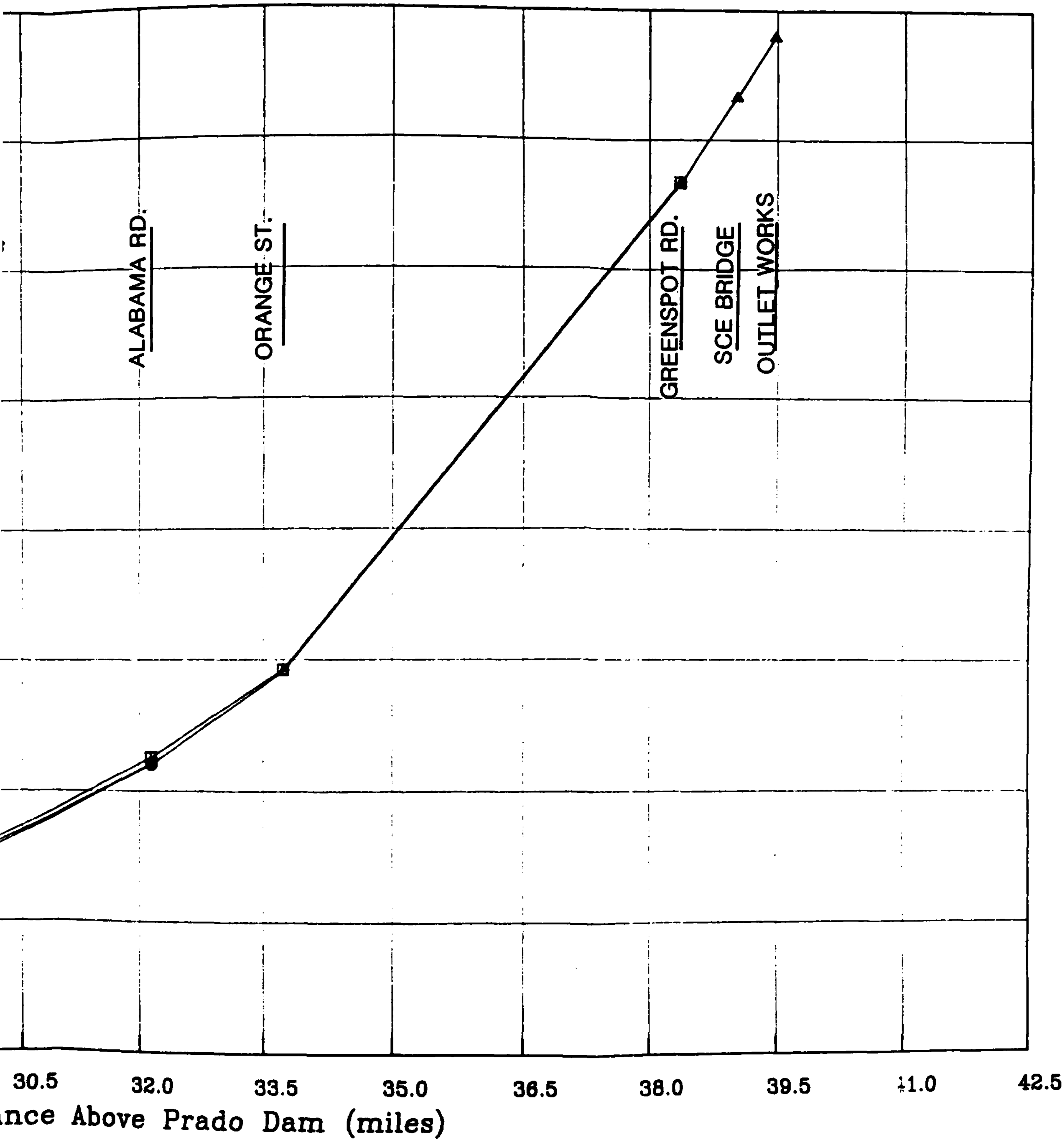


Figure 2.1. Continued.

District COE. Therefore, these gradation curves were disregarded. Three sediment-gradation curves, for samples collected downstream of Greenspot Road, were developed by SLA (March 1984). The existing bed material in this area was judged to still be reasonably represented by these samples. In January 1987, SLA collected six additional fine bed-material samples and 12 coarse bed-material samples for this study. All applicable bed-material gradation curves are given in Appendix A. A profile showing the variation in bed-material size along the study reach was developed and is tabulated in Table 2.2. As shown in the table, the bed material ranges from large cobbles and boulders near the dam site to medium sand in the lower reaches. In analyzing the bed-material size distributions, four representative bed-material reaches were identified. As shown in Figure 2.2, Reach A, extending from RM 39.5 to RM 38.5, consists primarily of boulders, cobbles, and coarse gravel. Reach B, extending from RM 38.5 to RM 33, consists primarily of cobbles, gravel, and coarse sand. Reach C, extending from RM 33 to RM 15.5, consists almost entirely of gravel and sand. The lowermost reach, Reach D, extends from RM 15.5 to RM 4 and is composed almost entirely of sand. Figures 2.3 to 2.6 present the representative bed-material size distributions for each of the four reaches.

2.4 Hydrology

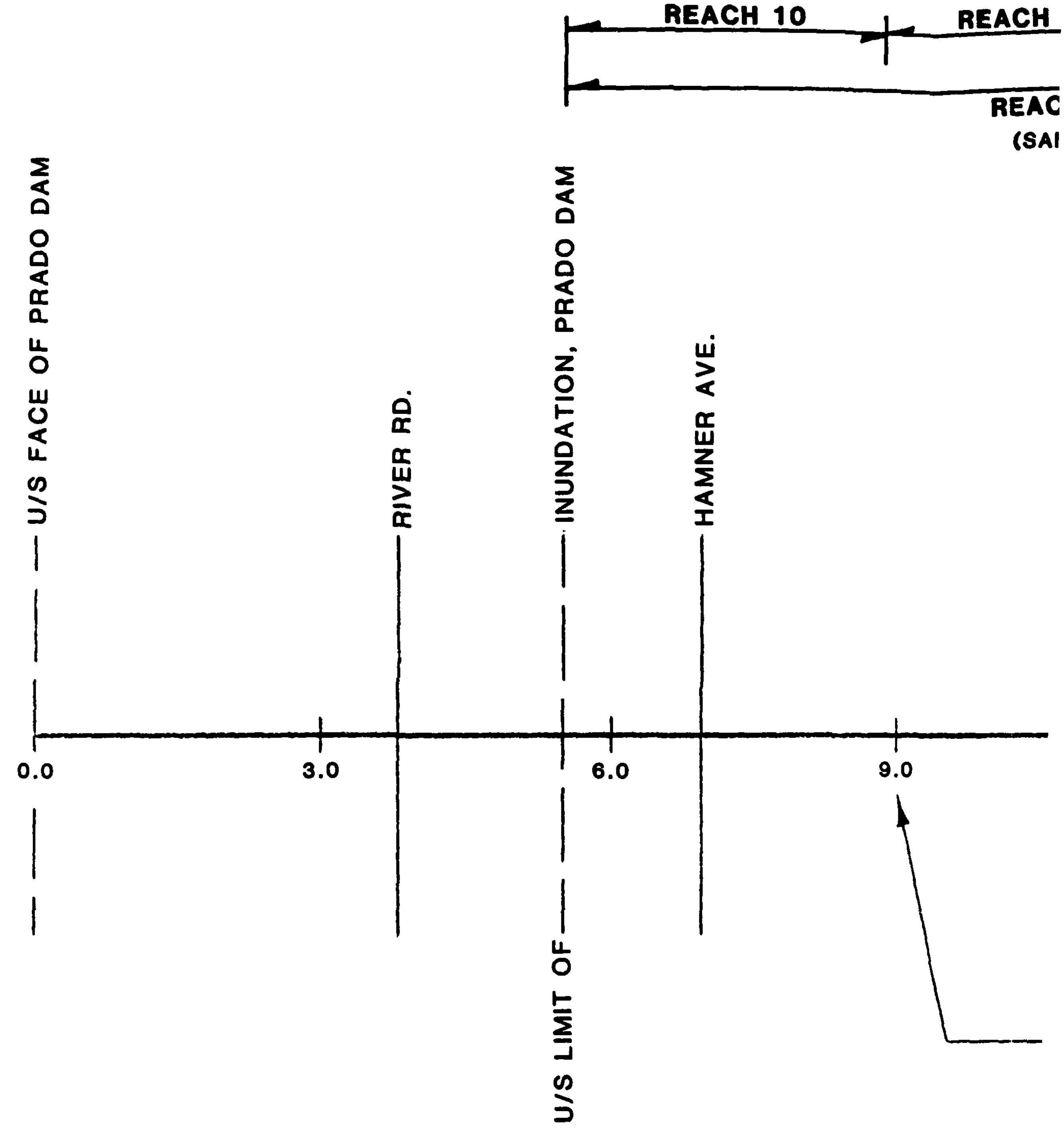
The pre- and postproject hydrographs and the proposed operating schedule for Seven Oaks Dam were provided by the Los Angeles District COE in the form of Los Angeles District Flood Hydrograph Package (LADFHP) computer output. Peak discharges along the Santa Ana River were given for the SPF under both pre- and postproject conditions. These discharges were provided in the LADFHP computer output and are given in Table 2.3 for the 10 hydraulically similar reaches indicated in Figure 2.2. Also provided were discharge-frequency curves for various locations along the Santa Ana River. All hydrologic information had been developed for future watershed conditions.

2.5 Geotechnical Investigations

An extensive geotechnical investigation of the Santa Ana River Canyon, particularly in the vicinity of the Seven Oaks Dam site, was undertaken by the COE (1985). In February 1984, a field and laboratory investigation program was initiated which included (1) seismic refraction surveys, (2) exploratory

Table 2.2. Bed-Sediment Profiles for the Study Reach.

River Mile	Bed-Sediment Sizes		
	d_{16} (mm)	d_{50} (mm)	d_{84} (mm)
4.00	0.26	0.46	0.90
6.92	0.16	0.23	0.41
21.44	0.35	1.3	17
24.23	0.34	1.9	34
26.88	0.54	1.8	23
27.80	0.18	0.70	13
29.10	0.26	0.71	9.5
32.16	0.20	0.50	3.8
33.74	1.1	25	203
38.36	1.0	30	88
38.95	27	120	420
39.05	12	102	460
39.39	19	90	310



REACH 10
REACH
REAC
(SAI
U/S FACE OF PRADO DAM
RIVER RD.
INUNDATION, PRADO DAM
U/S LIMIT OF
HAMNER AVE.
0.0
3.0
6.0
9.0

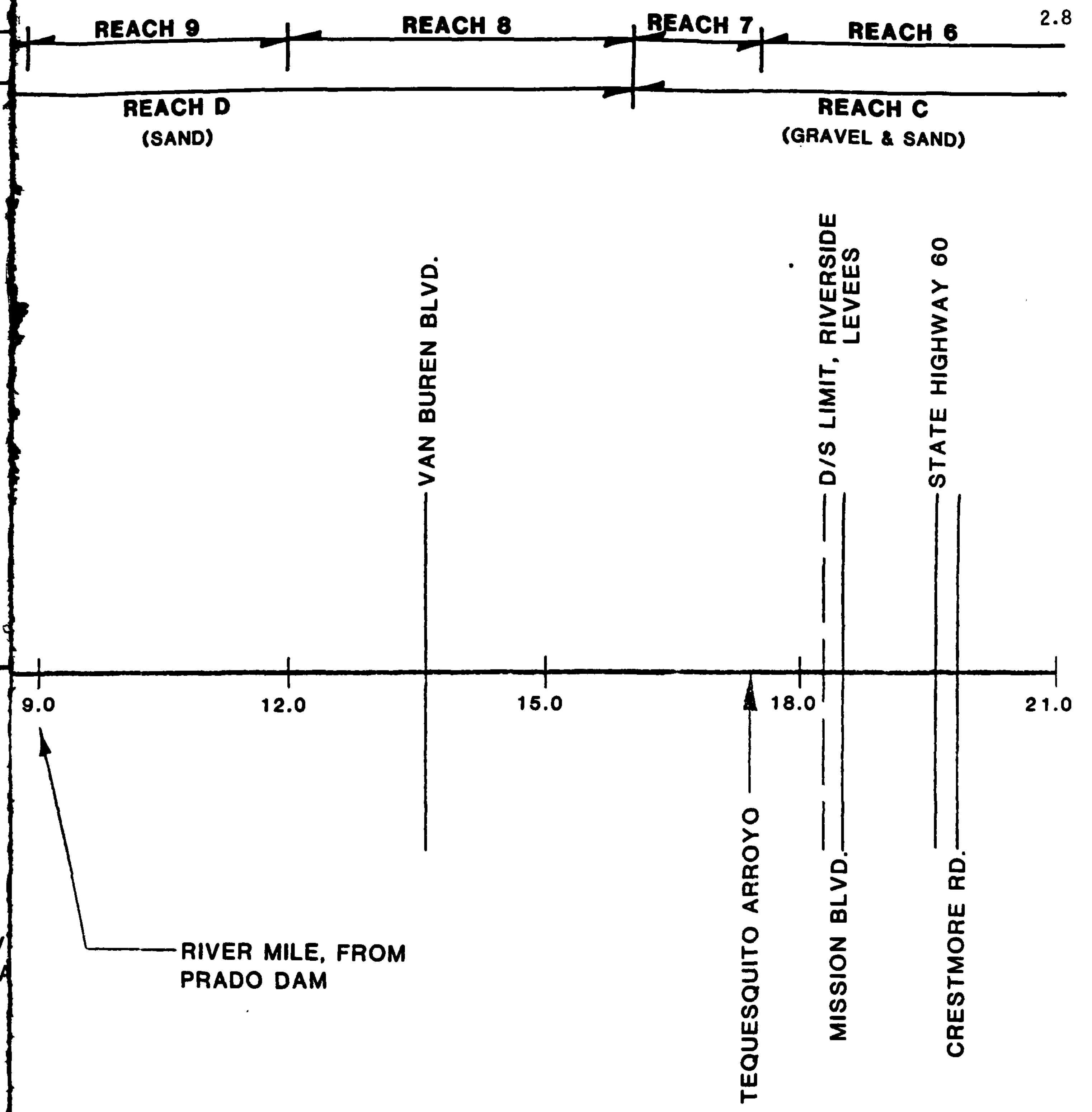
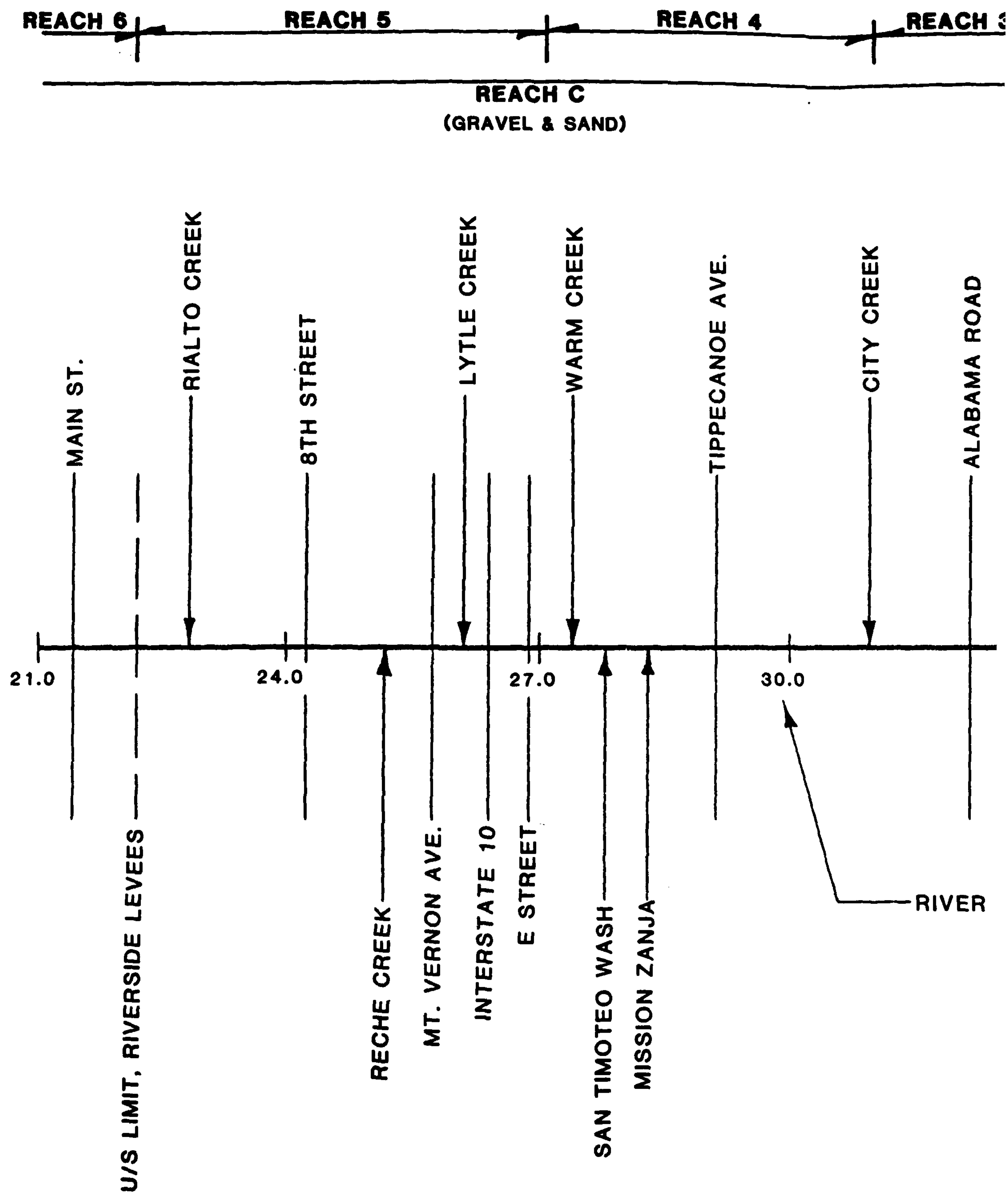


Figure 2.2.
SCHEMATIC OF ROAD CROSSINGS AND TRIBUTARIES ALONG THE SANTA ANA RIVER


REACH 6
REACH 5
REACH 4
REACH 3
REACH C
(GRAVEL & SAND)
MAIN ST.
RIALTO CREEK
8TH STREET
LYTLE CREEK
WARM CREEK
TIPPECANOE AVE.
CITY CREEK
ALABAMA ROAD
21.0
24.0
27.0
30.0
U/S LIMIT, RIVERSIDE LEVEES
RECHE CREEK
MT. VERNON AVE.
INTERSTATE 10
E STREET
SAN TIMOTEO WASH
MISSION ZANJA
RIVER

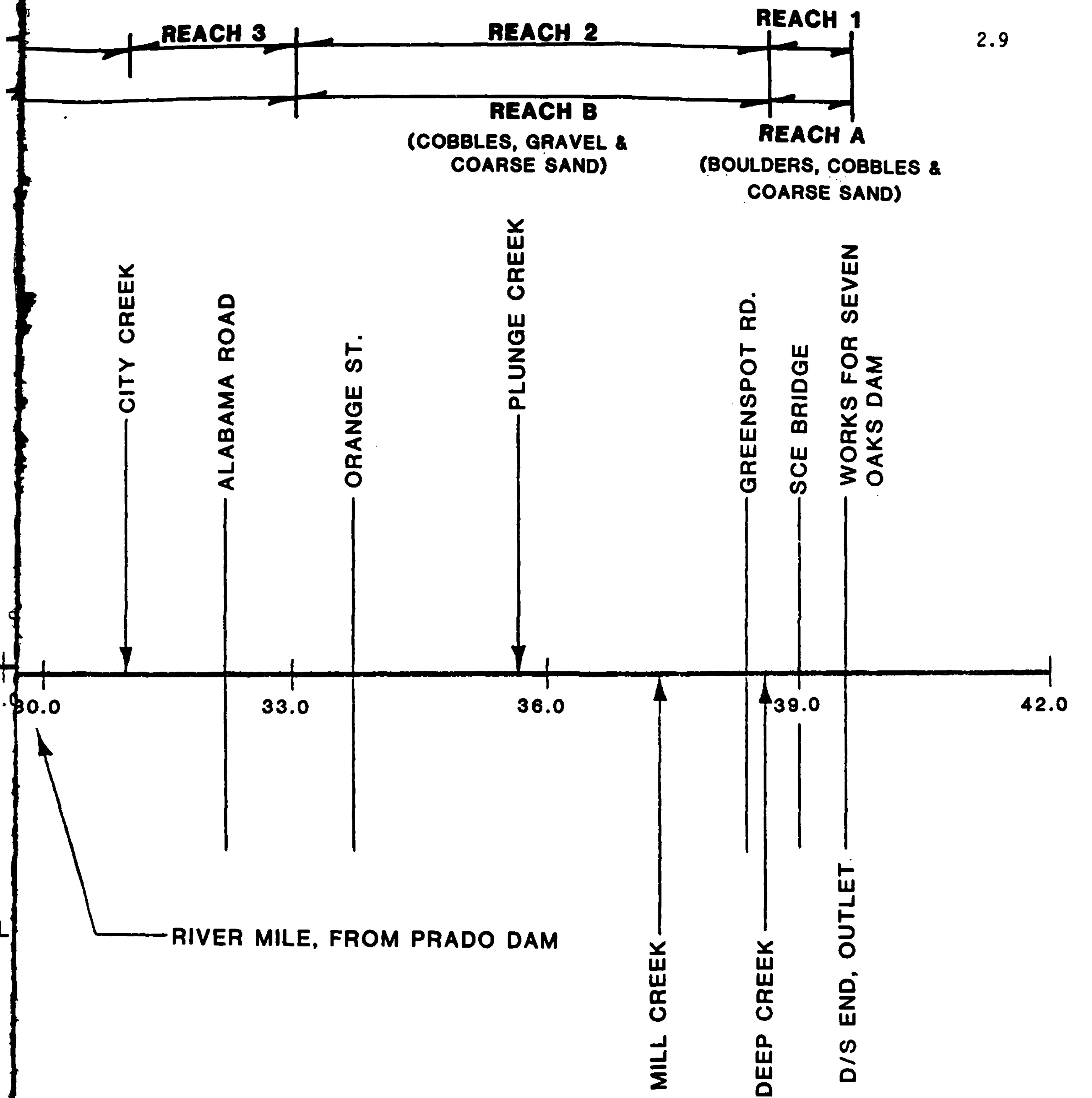


Figure 2.2.

SCHEMATIC OF ROAD CROSSINGS AND TRIBUTARIES ALONG THE SANTA ANA RIVER

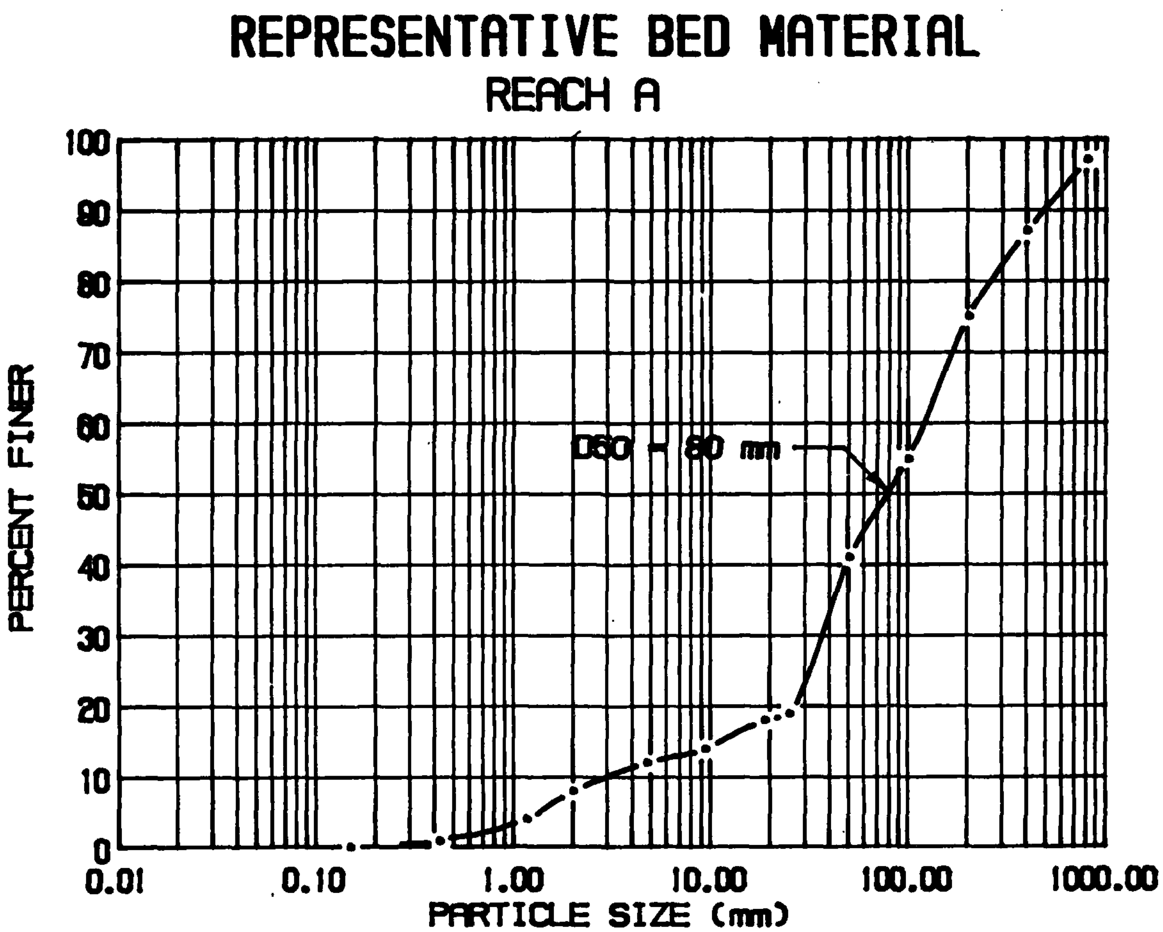


Figure 2.3. Representative bed material, Reach A, RM 38.5 to RM 39.5.

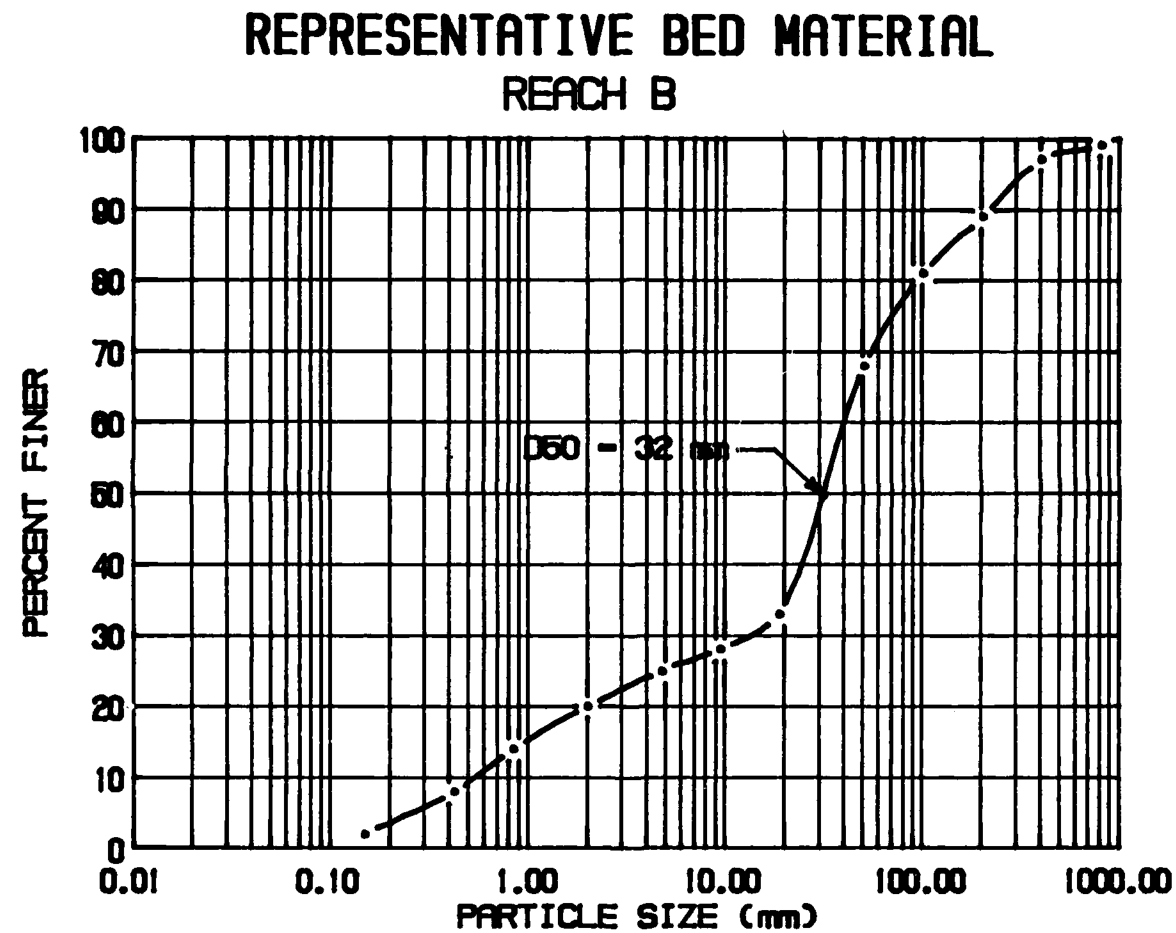


Figure 2.4. Representative bed material, Reach B, RM 33 to RM 38.5.

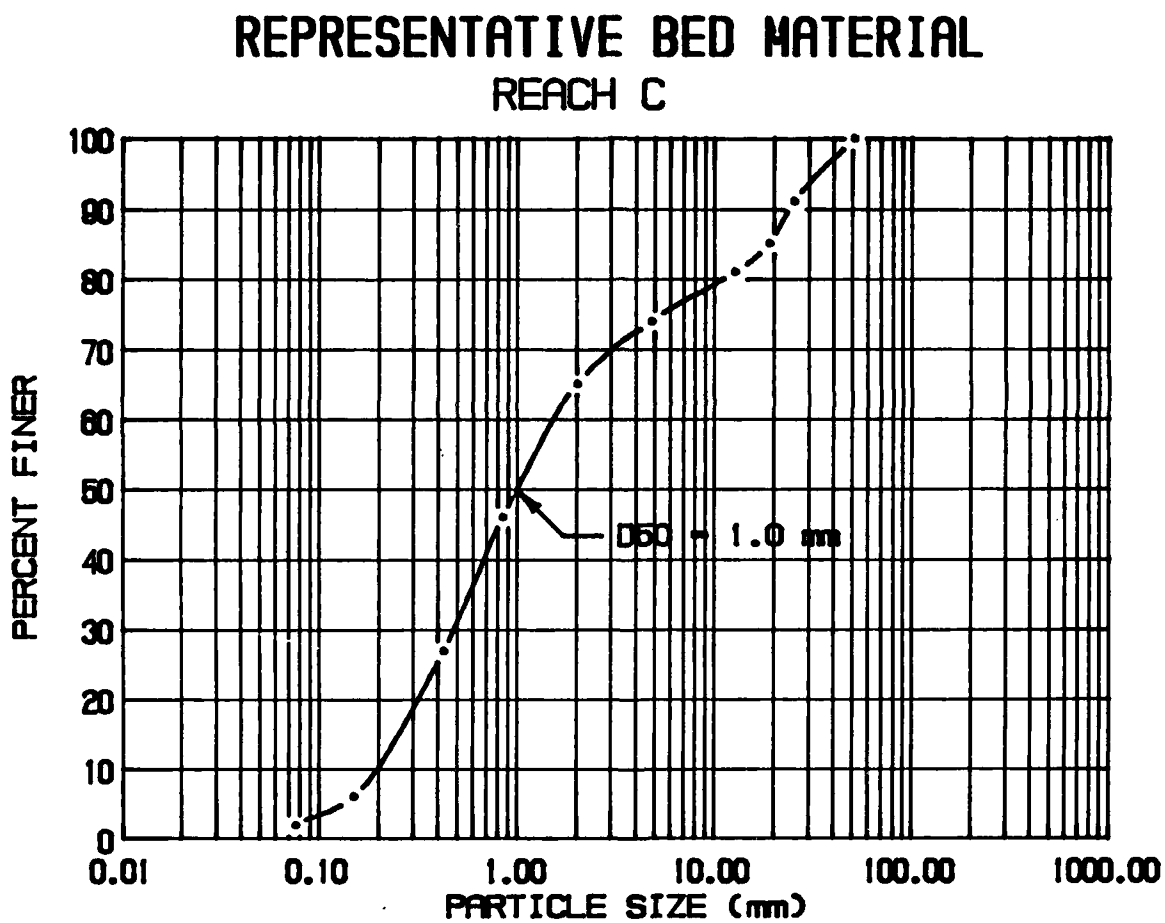


Figure 2.5. Representative bed material, Reach C, RM 15.5 to RM 33.

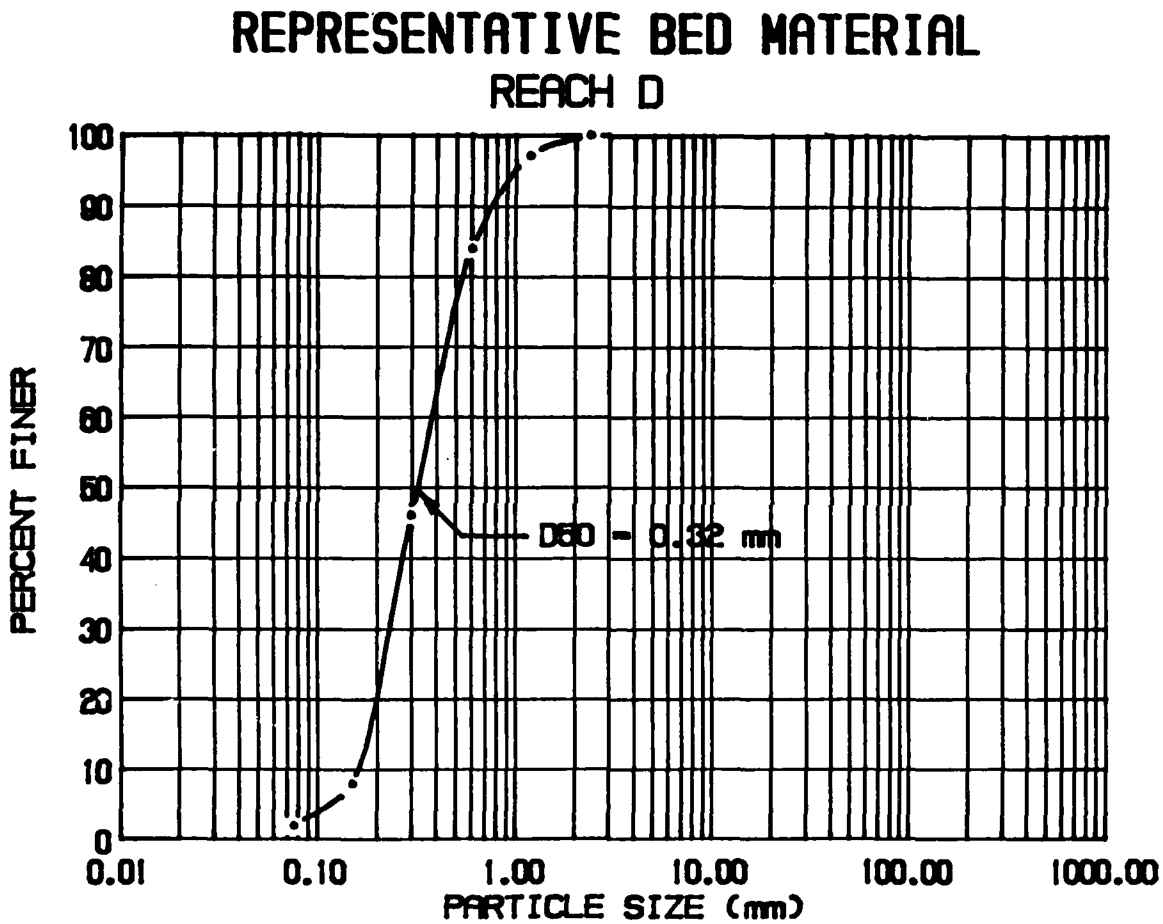


Figure 2.6. Representative bed material, Reach D, RM 4 to RM 15.5.

Table 2.3. Pre- and Postproject Future Condition SPF Peak Discharges (from LADFHP computer output).

Reach	Peak Discharge (cfs)	
	Preproject	Postproject
1	72,700	6,730
2-3	106,000	38,000
4	121,300	56,100
5-6	226,300	169,600
7-10	231,700	175,400

drilling and sampling, and (3) trenching. As a result of these investigations, the depth and configuration of bedrock in the canyon bottom was determined. It was found that the depth to top of bedrock, from the existing ground surface, varies from 30 to 110 feet. The configuration of bedrock under the dam site appears to be asymmetrical. Near the dam's centerline, the deepest erosional channel is at the east abutment. In general, the average depth to bedrock across the canyon is about 80 feet. Localized relief in the bedrock surface is probably 50 feet, and the deepest channels are about 100 feet below the existing ground surface. In the vicinity of the proposed outlet tunnel, bedrock is located approximately 100 feet below the existing channel bed (at about elevation 1,920 feet).

III. SEDIMENT-CONTINUITY ANALYSIS

The impact of Seven Oaks Dam on the Santa Ana River between the dam and Prado Reservoir was analyzed by evaluating the aggradation or degradation trends along the study reach using the sediment-continuity concept.

3.1 Identification of Representative Reaches

As previously discussed, bed-material samples were collected at several different sites along the Santa Ana River with the highest concentration of collection sites occurring near the upstream end of the study reach. In analyzing the bed-material size distributions, four representative bed-material reaches were identified (as shown in Figure 2.1). The bed-material size distributions for these representative reaches are given in Figures 2.3 to 2.6.

3.2 Hydraulic Analysis

The LADFHP computer output and COE discharge-frequency curves were used to determine pre- and postproject Santa Ana River discharges at several locations along the study reach for the 2-, 5-, 10-, 25-, 50-, and 100-year floods, as well as the SPF. Both pre- and postproject multiple profile HEC-2 analyses were performed using the discharge profiles. The results of the HEC-2 analyses provided the basis for further dividing the study reach into 10 computational reaches having similar hydraulic characteristics. The 10 hydraulically similar reaches are indicated in Figure 2.2. The results of the HEC-2 analyses are summarized on a reach-by-reach basis in Appendix B.

3.3 Development of Bed-Material Transport Equations

Bed-material transport relations of the form

$$q_s = a\, V^b\, Y^c \tag{3.1}$$

were derived for each of the four representative bed-material reaches where q_s is the bed-material transporting capacity in cfs per unit width, V is the mean channel velocity in fps, and Y is the mean hydraulic depth in feet. These relations were developed with the representative bed-material distributions and with the general range of hydraulic conditions of Reaches A through D using the Meyer-Peter, Muller Equation for bed load, and Einstein's integration of the suspended bed-material load. The general procedure involved com-

pution of bed-material transport rates for the anticipated range of hydraulic conditions followed by estimation of the coefficient a and exponents b and c (Equation 3.1) using multiple linear regression techniques. A detailed discussion of the Meyer-Peter, Muller Equation and Einstein's method is given in Appendix C. The results of this analysis are presented in Table 3.1.

After development of the bed-material transport relations, average hydraulic conditions for Reaches 1 through 10 for each of the return-period flows were used to develop rating curves of the form

$$Q_s = a\, Q^b \tag{3.2}$$

where Q_s is the bed-material transport rate in cfs and Q is the discharge in cfs for each of the 10 reaches. The resulting coefficient and exponent for the relations are summarized in Table 3.2.

3.4 Sediment-Continuity Calculations

The sediment characteristics of the Santa Ana River between the Seven Oaks Dam and Prado Reservoir were analyzed for both pre- and postproject conditions. The difference between the two conditions resulted primarily from the effect of the dam on the magnitude and shape of the flood-flow hydrographs along the river. For the postproject condition continuity analysis, it was assumed that channelization and channel stabilization measures would not be utilized.

The postproject hydrograph was broken into two segments in the analysis. The first segment covers the period of time during which the peak flows occur in the Santa Ana River downstream of the confluence with Mill Creek, and corresponds to the period during which the water levels in Prado Reservoir are rising. The second segment in the postproject hydrograph covers the period of time during which the peak releases from Seven Oaks Dam are occurring, and also corresponding to the period during which the levels in Prado Reservoir are falling. Figures 3.1 and 3.2 are example hydrographs displaying the preproject condition (Case 1), the postproject condition only while Prado Reservoir is rising (Case 2), and the postproject condition only while Prado Reservoir is falling (Case 3). The combination of the hydrographs for Cases 2 and 3 yields the entire postproject hydrograph. Table 3.3 lists the peak discharges from the representative hydrographs for each of the reaches.

Table 3.1. $q_s = av^by^c$ Relations.

Reach	a	b	c
A	6.94×10^{-6}	4.132	-0.194
B	1.97×10^{-5}	3.798	0.057
C	3.20×10^{-5}	3.431	0.066
D	2.02×10^{-5}	3.519	0.334

Table 3.2. $Q_s = aQ^b$ Relations.

Reach	a	b
1	3.93×10^{-4}	1.14
2	7.08×10^{-4}	1.16
3	3.67×10^{-4}	1.13
4	9.02×10^{-5}	1.22
5	8.23×10^{-5}	1.21
6	1.41×10^{-5}	1.20
7	2.34×10^{-3}	0.881
8	6.64×10^{-6}	1.47
9	3.52×10^{-5}	1.25
10	5.64×10^{-5}	1.18

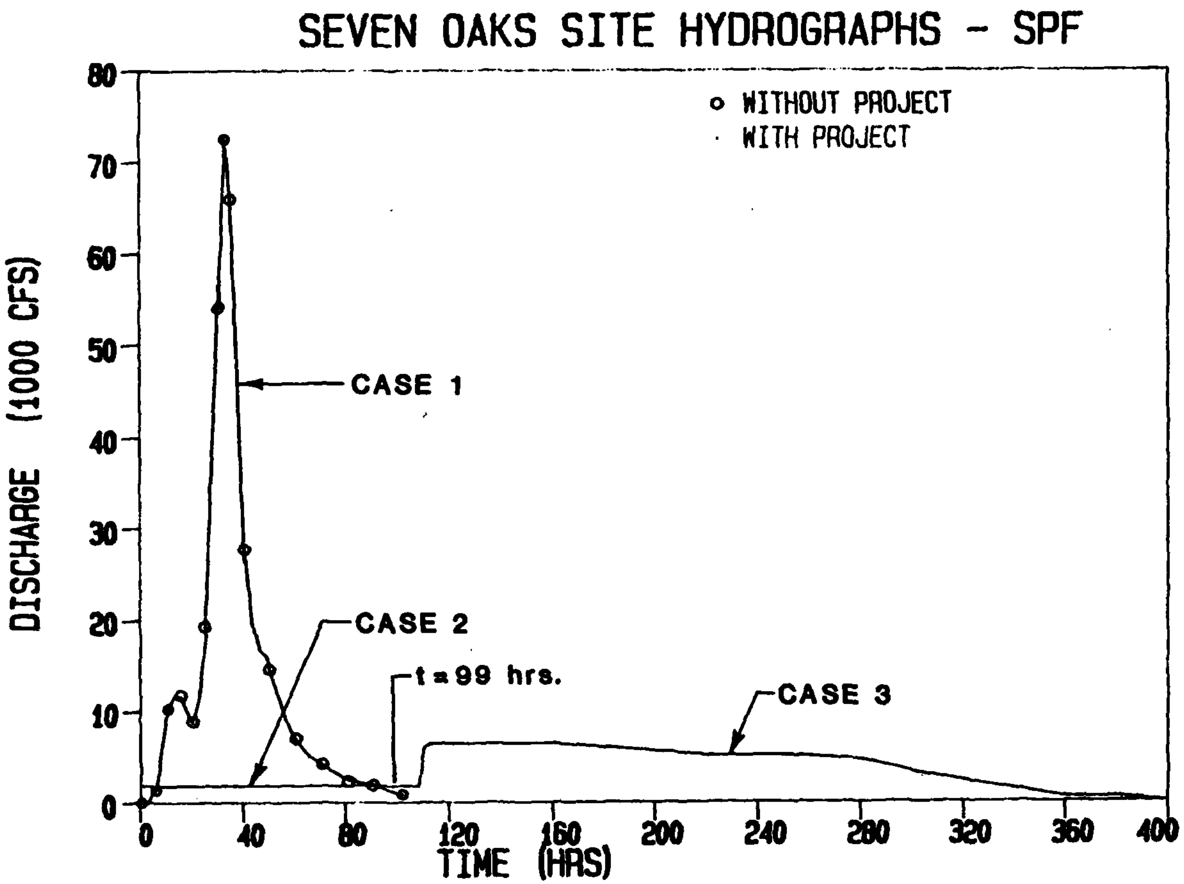


Figure 3.1. Example hydrographs displaying the three cases (see Section 3.3).

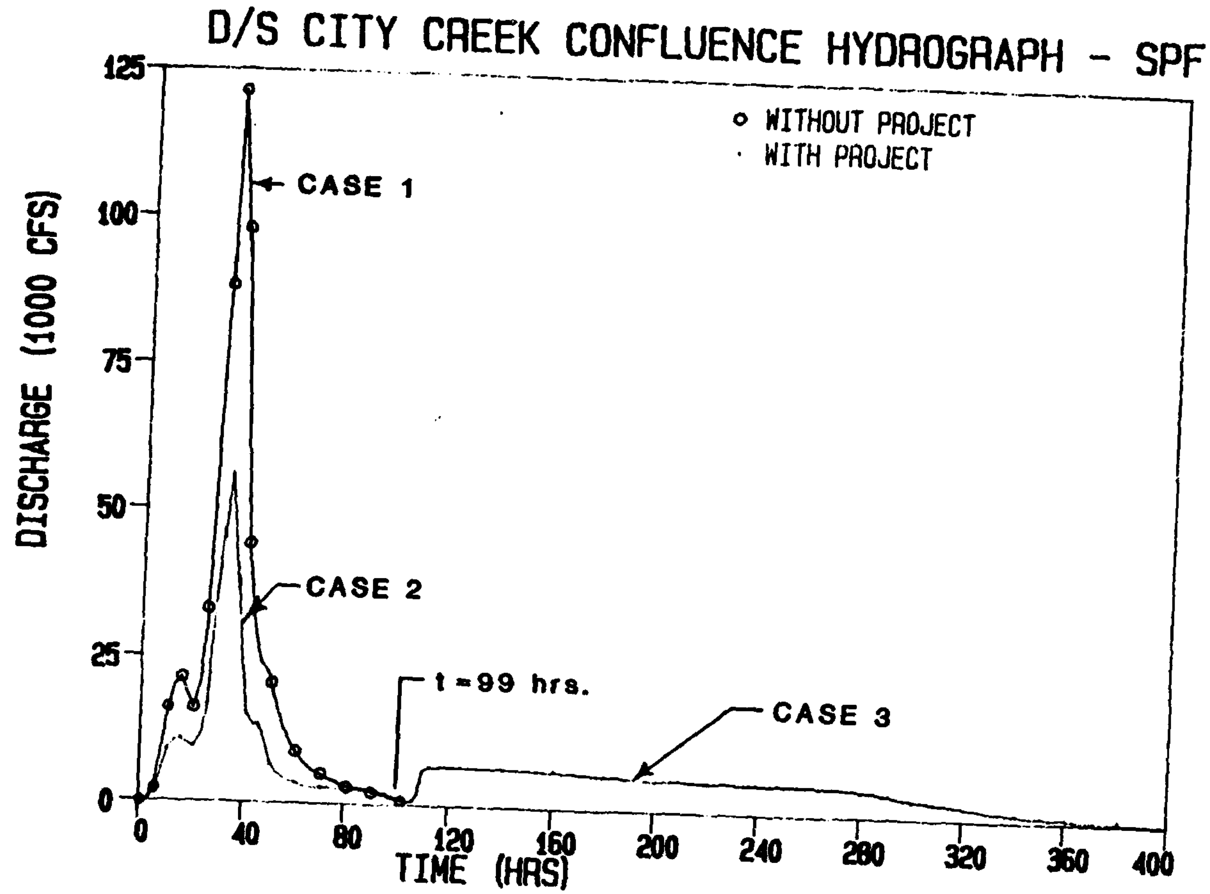


Figure 3.2. Example hydrographs displaying the three cases (see Section 3.3).

Table 3.3. Peak Santa Ana River Discharges for Each Case.

Reach	Flood Event	Case 1 Q_p (cfs)	Case 2 Q_p (cfs)	Case 3 Q_p (cfs)
1	SPF	72,700	2,000	6,730
	100-yr	59,500	2,000	5,950
	50-yr	36,300	2,000	5,180
	25-yr	21,700	2,000	3,890
	10-yr	9,600	916	2,460
	5-yr	4,500	41	1,910
	2-yr	1,100	0	530
2-3	SPF	106,000	38,000	6,730
	100-yr	77,600	29,600	5,950
	50-yr	45,500	20,100	5,180
	25-yr	26,700	13,100	3,890
	10-yr	12,000	6,700	2,460
	5-yr	5,700	3,600	1,910
	2-yr	1,500	1,100	530
4	SPF	121,300	56,100	6,730
	100-yr	82,000	32,200	5,950
	50-yr	48,000	21,000	5,180
	25-yr	28,000	13,300	3,890
	10-yr	12,400	6,700	2,460
	5-yr	6,000	3,600	1,910
	2-yr	1,600	1,150	530
5-6	SPF	226,300	169,600	6,730
	100-yr	180,900	135,300	5,950
	50-yr	110,900	79,700	5,180
	25-yr	60,800	44,700	3,890
	10-yr	24,600	18,200	2,460
	5-yr	10,600	7,800	1,910
	2-yr	2,100	1,600	530
7-10	SPF	231,700	175,400	6,730
	100-yr	187,100	141,000	5,950
	50-yr	115,000	83,000	5,180
	25-yr	63,000	46,500	3,890
	10-yr	25,500	19,000	2,460
	5-yr	11,000	8,100	1,910
	2-yr	2,200	1,650	530

Case 1: Preproject
Case 2: Postproject while Prado Reservoir is rising
Case 3: Postproject while Prado Reservoir is falling

Estimation of net scour/deposition depths was accomplished by applying the continuity principle

$$\text{Agg/Deg Rate} = Q_{s_{OUT}} - Q_{s_{IN}} \qquad (3.3)$$

where $Q_{s_{IN}}$ represents the supply of sediment into a channel reach and $Q_{s_{OUT}}$ represents the bed-material transport rate of the reach. The sediment supply contributed by tributaries was assumed to be the same for both pre- and postproject conditions. Since inadequate tributary sediment data were available, for comparison purposes, tributary sediment inflow was neglected. Using accurate tributary sediment data to predict supply may slightly change the net scour/deposition volumes in the reaches, but would not affect the comparative differences between pre- and postproject conditions. Likewise, the future construction of dams and debris basins by local agencies on tributary channels will have the same net scour/deposition effects regardless of construction of Seven Oaks Dam.

The volume of sediment transported by an individual reach, for a particular event, was estimated by converting the appropriate water-discharge hydrograph to a sediment-discharge hydrograph using the sediment discharge-water discharge relations given in Table 3.2, and integrating the resulting sediment-discharge hydrograph over the event's duration. The volume of sediment carried by an individual reach was considered the upstream sediment load for the next downstream reach. The net sediment volume deposited in or taken from a reach was computed as the difference between the volume of the upstream sediment load and the volume of sediment the reach is capable of transporting. The volume of entrained air in the soil was accounted for by applying a bulking factor of 1.4 to the net sediment volume. This bulking factor is typical for the sands and gravel present in the study reach. It was assumed that deposition or scour would occur uniformly over the entire reach. Therefore, the aggradation or degradation depth was estimated by dividing the bulked net sediment volume by the length and average effective width of the reach. These computations were performed for each reach for the seven flood events considered. The results are summarized in Appendix D.

After computing sediment-continuity depths, degradational reaches were analyzed to determine the armoring tendency before the continuity depth is reached. The armoring potential for reaches determined to be degradational in the sediment-continuity analysis was computed using Shields criteria for

incipient motion and a representative bed-material composition. Shields criteria for determining incipient motion is given by:

$$D_c = \frac{\tau}{0.047\ (\gamma_s - \gamma)} \tag{3.4}$$

where D_c is the diameter of the sediment particle at incipient motion, τ is the boundary shear stress, γ_s and γ are the specific weights of sediment and water, respectively, and 0.047 is a dimensionless coefficient referred to as the Shields parameter. Equation 3.4 was used to establish the incipient particle size for each reach for the various return period discharges. The depth of scour necessary to establish an armor layer (ΔZ_a) was then calculated from:

$$\Delta Z_a = Y_a\ (\frac{1}{P_c} - 1) \tag{3.5}$$

where Y_a is the thickness of the armor layer and P_c is the fraction of material coarser than the armor size (determined from the representative bed-material gradation). The thickness of the armor layer was taken to equal twice the diameter of the incipient particle size. Table 3.4 presents a comparison of the continuity derived scour depth and the armor depth for Reaches 1 and 2 for the standard project and 100-year floods. Continuity computed scour depths for Reach 1, Case 1 cannot be determined because the upstream sediment supply is unknown. For Cases 2 and 3, the upstream sediment supply to Reach 1 was assumed to be zero due to sediment retention by the proposed dam. The other degradational reaches did not armor for the return-period flows for any of the cases.

In addition, armor depths were calculated for the design release of 8,000 cfs and, as a worst case scenario, for the maximum possible release (conduit capacity) of 15,000 cfs from Seven Oaks Dam. The results of this computation are presented in Table 3.5.

Table 3.6 summarizes the sediment continuity and armoring analyses for the overall study reach. It can be seen from the Case 1 depths and the combined postproject depths that generally minor postproject scour/deposition impact can be expected. An exception occurs in Reach 7 where the scour potential increases from approximately 0.4 feet for the preproject condition to 1.5 feet for the postproject condition for the SPF. The increased scour is due in part to decreased upstream supply and also in part to the characteristics of the sediment-rating curve for that reach. Since the flood plain along Reach 7

Table 3.4. Comparison of Armor Depths with Sediment Continuity Depths for Reaches 1 and 2.

Case 1: Without project condition.
Case 2: With project condition for Seven Oaks releases, while Prado Reservoir is rising only.
Case 3: With project condition for peak Seven Oaks releases, while Prado Reservoir is falling only.

Table 3.4a. Case 1 Armor/Continuity Comparison.

Reach	Flood Event	Armor Depth (ft)	Continuity Scour Depth (ft)	Net Scour Depth* (ft)
1	SPF	7.8	--	--
(Supply)	100	5.7	--	--
2	SPF	9.1	1.1	1.1
	100	7.5	0.8	0.8

Table 3.4b. Case 2 Armor/Continuity Comparison.

Reach	Flood Event	Armor Depth (ft)	Continuity Scour Depth (ft)	Net Scour Depth* (ft)
1	SPF	0.2	2.7	0.2
	100	0.2	2.7	0.2
2	SPF	2.1	0.9	0.9
	100	1.8	0.7	0.7

Table 3.4c. Case 3 Armor/Continuity Comparison.

Reach	Flood Event	Armor Depth (ft)	Continuity Scour Depth (ft)	Net Scour Depth* (ft)
1	SPF	0.4	9.8	0.4
	100	0.4	4.7	0.4
2	SPF	0.6	1.4	0.6
	100	0.5	0.6	0.5

*Net scour is the smaller of either the armor depth or the continuity scour depth.

Table 3.5. Armor Depths for Maximum Seven Oaks Releases.

Reach	Discharge (cfs)	Armor Depth (ft)
1	8,000	0.5
	15,000	0.8
2	8,000	0.9
	15,000	1.8

Table 3.6. Santa Ana River Deposition/Scour Depths (+ indicates deposition).

Reach	Flood Event	Case 1 Depth (ft)	Case 2 Depth (ft)	Case 3 Depth (ft)	Combined Postproject Depth, Cases 2 + 3 (ft)	Difference in Depth (Cases 2 + 3 - Case 1) (ft)
1	SPF	--	-0.2	-0.4	-0.6	--
	100	--	-0.2	-0.4	-0.6	--
	Avg. Ann.	--	--	--	--	--
2	SPF	-1.1	-0.9	-0.6	-1.4	-0.3
	100	-0.8	-0.7	-0.5	-1.2	-0.4
	Avg. Ann.	-0.15	-0.11	-0.05	-0.16	-0.01
3	SPF	+1.1	+0.4	+0.0	0.5	-0.6
	100	0.8	0.3	0.2	0.5	+0.2
	Avg. Ann.	0.09	0.05	0.03	0.08	-0.01
4	SPF	+0.2	+0.1	+0.3	+0.4	+0.2
	100	0.2	0.1	0.1	0.3	+0.1
	Avg. Ann.	0.04	0.03	0.02	0.05	+0.01
5	SPF	-0.2	-0.3	+0.0	-0.3	-0.1
	100	-0.3	-0.3	0.0	-0.3	0.0
	Avg. Ann.	-0.03	-0.03	0.00	-0.03	0.00
6	SPF	+0.6	+0.5	+0.2	+0.7	+0.1
	100	0.5	0.4	0.1	0.5	0.0
	Avg. Ann.	0.07	0.05	0.01	0.06	-0.01
7	SPF	-0.4	-0.3	-1.1	-1.5	-1.1
	100	-0.4	-0.3	-0.5	-0.8	-0.4
	Avg. Ann.	-0.18	-0.16	-0.18	-0.34	-0.16
8	SPF	-1.1	-0.7	+0.5	-0.2	+0.9
	100	-0.8	-0.5	0.2	-0.3	+0.5
	Avg. Ann.	-0.02	0.01	0.07	0.08	+0.10
9	SPF	+0.5	+0.3	+0.0	+0.3	-0.2
	100	0.4	0.2	0.0	0.2	-0.2
	Avg. Ann.	0.04	0.02	0.00	0.02	-0.02
10	SPF	+0.2	+0.1	+0.0	+0.1	-0.1
	100	0.1	0.1	0.0	0.1	0.0
	Avg. Ann.	0.03	0.02	0.00	0.02	-0.01

has experienced little development, and there are no bridge crossings within the reach, the adverse impacts of the increased scour potential of 1.1 feet for the SPF may be negligible.

Also included in Table 3.6 are the average annual scour/deposition depths for the three cases. For each reach, the average annual sediment volume was computed by integrating the reach's probability versus sediment volume curve. These curves were developed from the sediment transport analysis results for individual events for each reach. The integration equation was developed based on the SPF corresponding to approximately a 180-year event. The equation for the average annual sediment volume (V_{AA}) is:

$$V_{AA} = 0.0078\ V_{SPF} + 0.0072\ V_{100} + 0.015\ V_{50} + 0.040\ V_{25} + 0.230\ V_{10} + 0.450\ V_2 \tag{3.6}$$

where V_{SPF}, V_{100}, ..., V_2 are sediment volumes for individual events. Sediment continuity concepts were then used to determine net scour or deposition volumes for the reaches. A bulking factor of 1.4 was applied to the net scour/deposition volumes. The average annual scour or deposition depths were then computed by assuming scour or deposition is uniformly distributed over each reach. The resulting average annual scour and deposition depths are approximately equal to those determined for the 10-year event. The average annual deposition and scour depths given in Table 3.6 represent short-term rates of aggradation and degradation. Over time these rates will decrease as the channel tends toward an equilibrium bed slope. Furthermore, for Reaches 1 and 2, the armoring potential is high. Therefore, these reaches will eventually armor which would completely end the degradation process in these reaches. As with the SPF and 100-year depths, the difference between average annual scour/deposition depths for pre- and postproject conditions are minor. Since the maximum difference in average annual scour/deposition depth between pre- and postproject conditions for any reach is less than 0.2 feet, the impact of the dam may not be significant.

3.5 Conclusions

The long-term impacts of Seven Oaks Dam on downstream deposition or scour can be expected to be slight. For the standard project flood, in almost all of the reaches, the effect of the postproject hydrograph is almost negligible. Due to the reduced flows, the armoring potential in Reach 1 is significantly

higher than for preproject conditions indicating that little or no degradation is likely to occur. Reach 7 may experience an increase in scour potential (an approximately 1.1-foot increase for the SPF. Unlike the upper reaches, the scour potential in Reach 7 will not be limited by armoring. Since the reach contains no significant structures, flood plain development is relatively minor and the increased scour potential is relatively small, this may not be significant; however, detailed investigations of local conditions should confirm this conclusion. Expected deposition in Reaches 3 and 6 may have a slight impact on the SPF and 100-year water-surface profiles.

Subsequent to the completion of the sediment-transport analysis, the proposed Seven Oaks Dam operating schedule was revised to limit the Case 2 releases to 500 cfs. Consequently, the maximum discharge in Reach 1 for Case 2 conditions would be 500 cfs, rather than the previously established 2,000 cfs. This flow reduction would decrease the sediment transported in Reach 1 and, therefore, the upstream sediment load for Reach 2. This would result in a slight increase in scour potential for Reach 2. However, the increase may not be substantial due to the relatively high armoring potential in Reach 2.

IV. FORMULATION AND EVALUATION OF CONCEPTUAL ALTERNATIVE COMPONENTS

To minimize the adverse impacts to the downstream channel which would result from construction of Seven Oaks Dam and subsequent reservoir releases, 12 alternative components were formulated. The alternative components include several alternatives for energy dissipation downstream of the proposed outlet portal exit channel and alternatives for channel stabilization upstream of Greenspot Road. Performance criteria were developed to evaluate the effectiveness of the alternative components in alleviating adverse impacts. The 12 alternative components were evaluated based on their ability to satisfy the performance criteria and to minimize project costs. Three were selected for more detailed study.

4.1 Criteria for Alternative Component Evaluation

Three types of criteria were considered in the formulation and evaluation of the alternative components. They were (1) design, (2) performance, and (3) cost criteria.

4.1.1 Design Criteria

The criteria used to design the alternative components were supplied by four sources. The Corps of Engineers (COE) Engineering Manual 1110-2-1602, "Hydraulic Design of Reservoir Outlet Works," 1980, was utilized in designing the stilling basin. Engineering Manual 1110-2-1601, "Hydraulic Design of Flood Control Channels," 1970, was used to design the downstream channelization. In addition, Engineering Monograph No. 25, "Hydraulic Design of Stilling Basins and Energy Dissipators," published by the U.S. Department of the Interior, Bureau of Reclamation, 1978, was used in designing the submerged bucket and flip bucket alternatives, and provided supplemental information for the stilling basin design. Finally, Bureau of Reclamation publication, "Design of Small Dams," 1974, provided supplemental guidance in designing the alternative components.

4.1.2 Performance Criteria

The performance criteria were developed so that the effectiveness of the alternative components, in alleviating project induced adverse impacts, could be evaluated. In the following discussion, damages and other adverse impacts refer only to those induced by construction of the project. The criteria

included a list of objectives that the selected design alternative should satisfy. The criteria are as follows:

1. Protection of Seven Oaks Dam embankment against damage due to scour.

2. Protection of the outlet portal exit channel against damage due to scour.

3. Protection of Santa Ana River #3 powerhouse, flume, and other Southern California Edison (SCE) structures (e.g., bridge and buildings) from inundation and damage due to scour.

4. Protection of San Bernardino County Water Conservation District's canal headworks and cobble weir, both located at the SCE bridge, against damage due to scour.

5. Protection of Greenspot Pipeline, Redlands Aqueduct, Morton Canyon Connector, Bear Valley Highline, and Redlands Tunnel against damage due to scour.

6. Protection of the Greenspot Road bridge from damage due to high flows and scour.

7. Stabilization of the channel bed.

8. Protection of the canyon walls against slope failure.

9. Minimization of the impact to the endangered species (Eriastrum) in Upper Santa Ana Canyon.

10. Minimization of maintenance efforts.

4.1.3 Cost Criteria

Construction costs were also considered when evaluating the alternative components. The selected design alternative should combine a technically feasible configuration which meets all of the performance criteria while minimizing construction and maintenance costs. However, performance should not be unduly compromised for a less expensive design alternative.

4.2 Conceptual Alternative Components

The alternative components investigated to mitigate the adverse impacts caused by Seven Oaks Dam range from doing nothing to constructing sizable energy dissipators. The 12 conceptual alternatives considered are as follows:

1. *No energy dissipator or channelization.*

2. Preformed plunge pool and toe protection for the outlet channel.
3. Stilling basin.
4. Stilling basin with channel stabilizer.
5. Submerged slotted bucket.
6. Flip bucket.
7. Flip bucket with preformed plunge pool and toe protection for the outlet channel.
8. Extend toe protection for proposed dam embankment to bedrock.
9. Channelization upstream of Greenspot Road.
10. Drop structures.
11. Site-specific erosion/flood-control protection.
12. Minimizing the impact to the endangered species.

4.2.1 Alternative 1: No Energy Dissipator or Channelization

For the no energy dissipator or channelization alternative, the reservoir releases would flow directly out of the outlet portal exit channel and impact the existing ground relatively near the end of the outlet. Although this alternative would cost nothing, it would provide no energy dissipation until a natural scour hole was formed. Additionally, no protection of the outlet portal exit channel from damage due to scour would be provided. Material eroded during formation of the scour hole would be deposited in the downstream channel.

4.2.2 Alternative 2: Preformed Plunge Pool and Toe Protection for the Outlet Portal Exit Channel

Adding a preformed plunge pool and toe protection for the outlet portal exit channel to Alternative 1 would protect the outlet portal exit channel with a concrete cutoff wall at the end of the outlet, provide energy dissipation with the preformed plunge pool, and eliminate most of the deposition associated with natural formation of the scour hole. Further, this alternative would be cost effective, requiring only excavation and construction of a cutoff wall. However, since releases would simply exit the outlet portal exit channel, the plunge pool would be located relatively close to the end of the outlet tunnel requiring a relatively deep cutoff wall. Determination of

the potential depth of scour and the scour hole configuration for this alternative are explained in Appendix E. The plan view of this alternative is shown on Sheet 3.

For this alternative, and all others where a scour hole is formed, there are two processes by which flows can leave the scour hole. While water is being released from the reservoir, the primary mode of escape would be overflow of the scour hole. The scour hole configuration presented in Appendix E assumes that overflow protection for the scour hole would not be provided. Groundwater absorption is other mode by which flows could escape but would be possible only when the groundwater table is lower than the water level in the scour hole.

4.2.3 Alternative 3: Stilling Basin

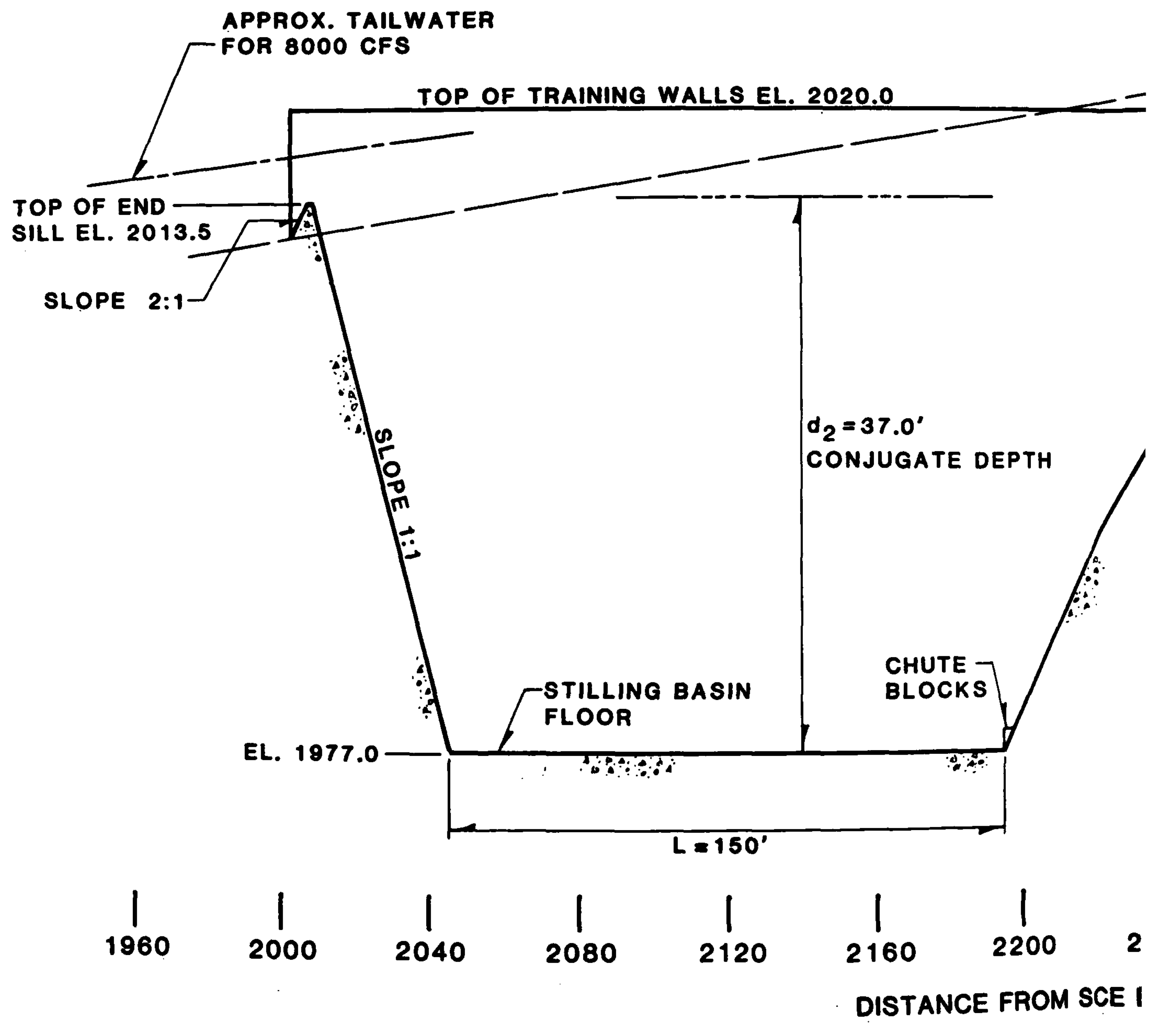
The stilling basin alternative would provide adequate energy dissipation, but would require a basin approximately 200 feet long, 45 feet wide, and, due to the lack of adequate tailwater, 43 feet deep. Constructing a concrete basin to these dimensions would be prohibitively expensive. Furthermore, a stilling basin is not a preferred method for dissipating energy when considering outlet velocities greater than 100 feet per second.

4.2.4 Alternative 4: Stilling Basin with Channel Stabilizer

The stilling basin with channel stabilizer would simply be the stilling basin described above with a cutoff wall located at its downstream end to isolate scour in the downstream channel. The plan view of this alternative is shown on Sheet 4 and the profile and detail sketches are given in Figures 4.1 and 4.2, respectively. The advantages and disadvantages for the "stilling basin" alternative also apply to this alternative. In addition, because the existing channel bed is relatively stable, the channel stabilizer would add to the cost of construction without providing significant benefits. The existing channel is armored which will limit the potential for degradation.

4.2.5 Alternative 5: Submerged Slotted Bucket

The submerged slotted bucket alternative would provide energy dissipation, but would require that the bucket invert be located approximately 84 feet below existing ground. The plan view of this alternative is shown on Sheet 5 and the profile and detail are given in Figures 4.3 and 4.4, respec-



APPROX. TAILWATER
FOR 8000 CFS
TOP OF TRAINING WALLS EL. 2020.0
TOP OF END
SILL EL. 2013.5
SLOPE 2:1
SLOPE 1:1
d_2 = 37.0'
CONJUGATE DEPTH
CHUTE
BLOCKS
STILLING BASIN
FLOOR
EL. 1977.0
L = 150'
1960
2000
2040
2080
2120
2160
2200
2
DISTANCE FROM SCE

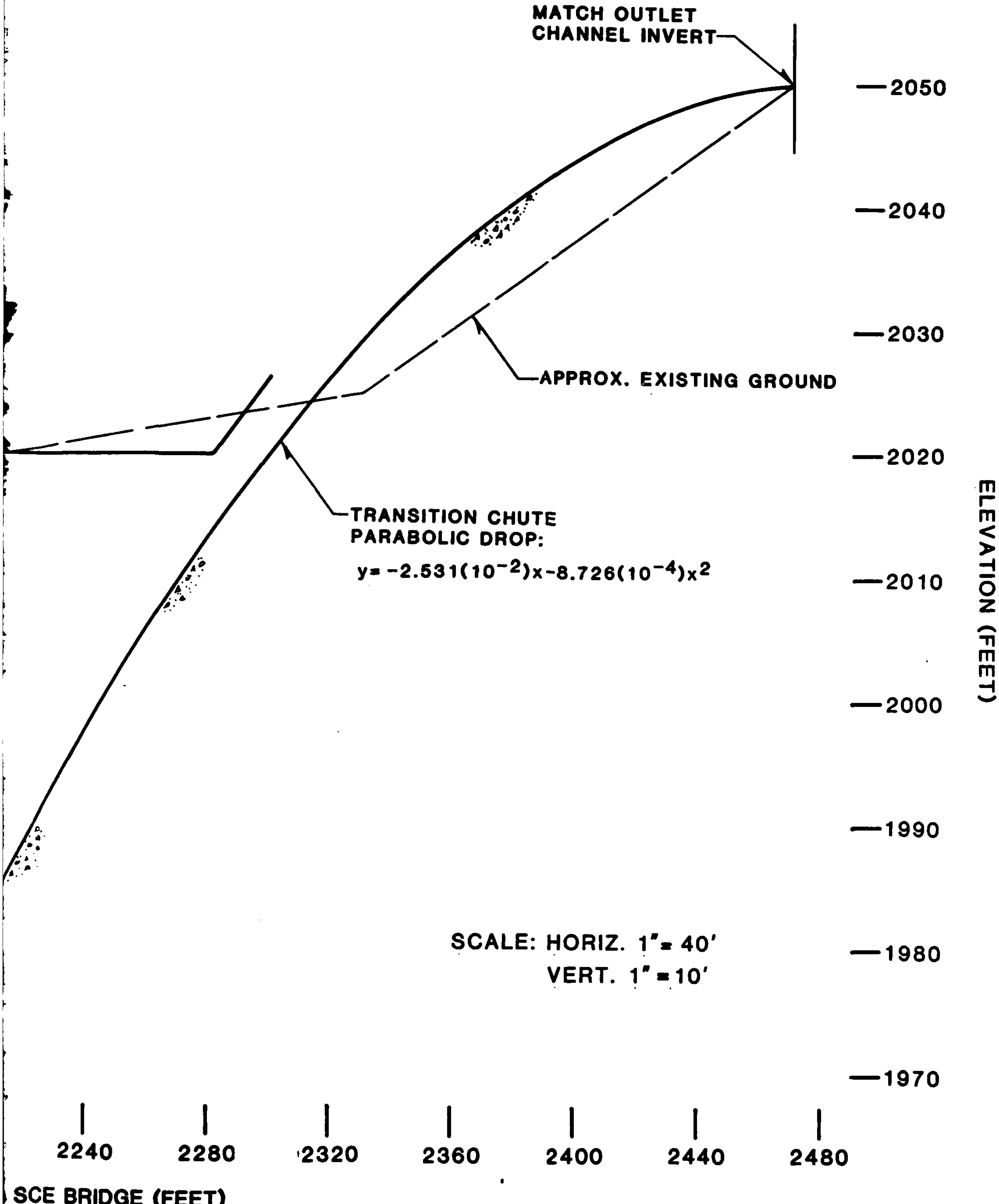
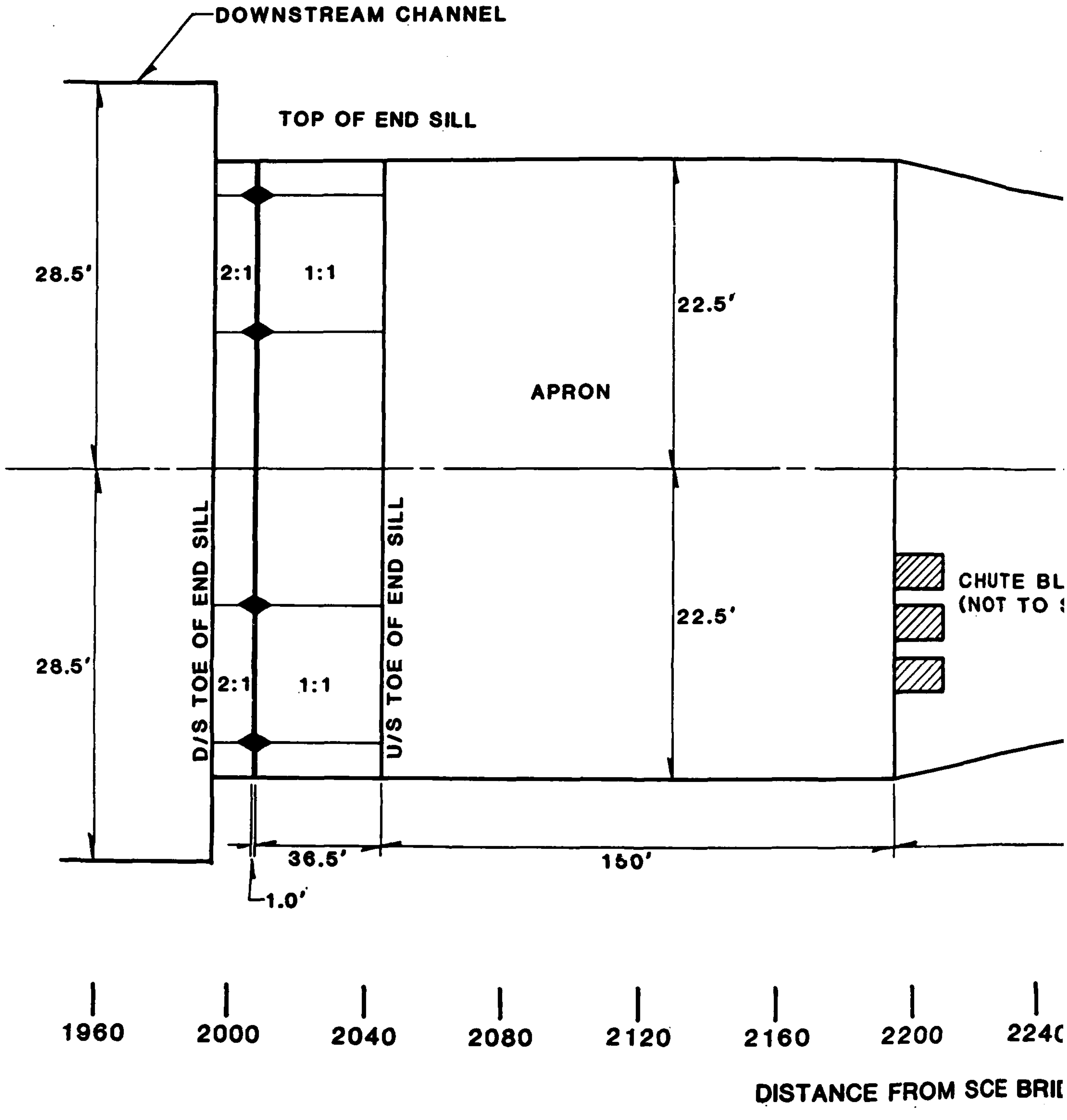


Figure 4.1. Profile of stilling basin.



DOWNSTREAM CHANNEL
TOP OF END SILL
28.5'
2:1
1:1
22.5'
APRON
D/S TOE OF END SILL
U/S TOE OF END SILL
CHUTE BL
(NOT TO
22.5'
28.5'
2:1
1:1
36.5'
150'
1.0'
1960
2000
2040
2080
2120
2160
2200
224
DISTANCE FROM SCE BRI

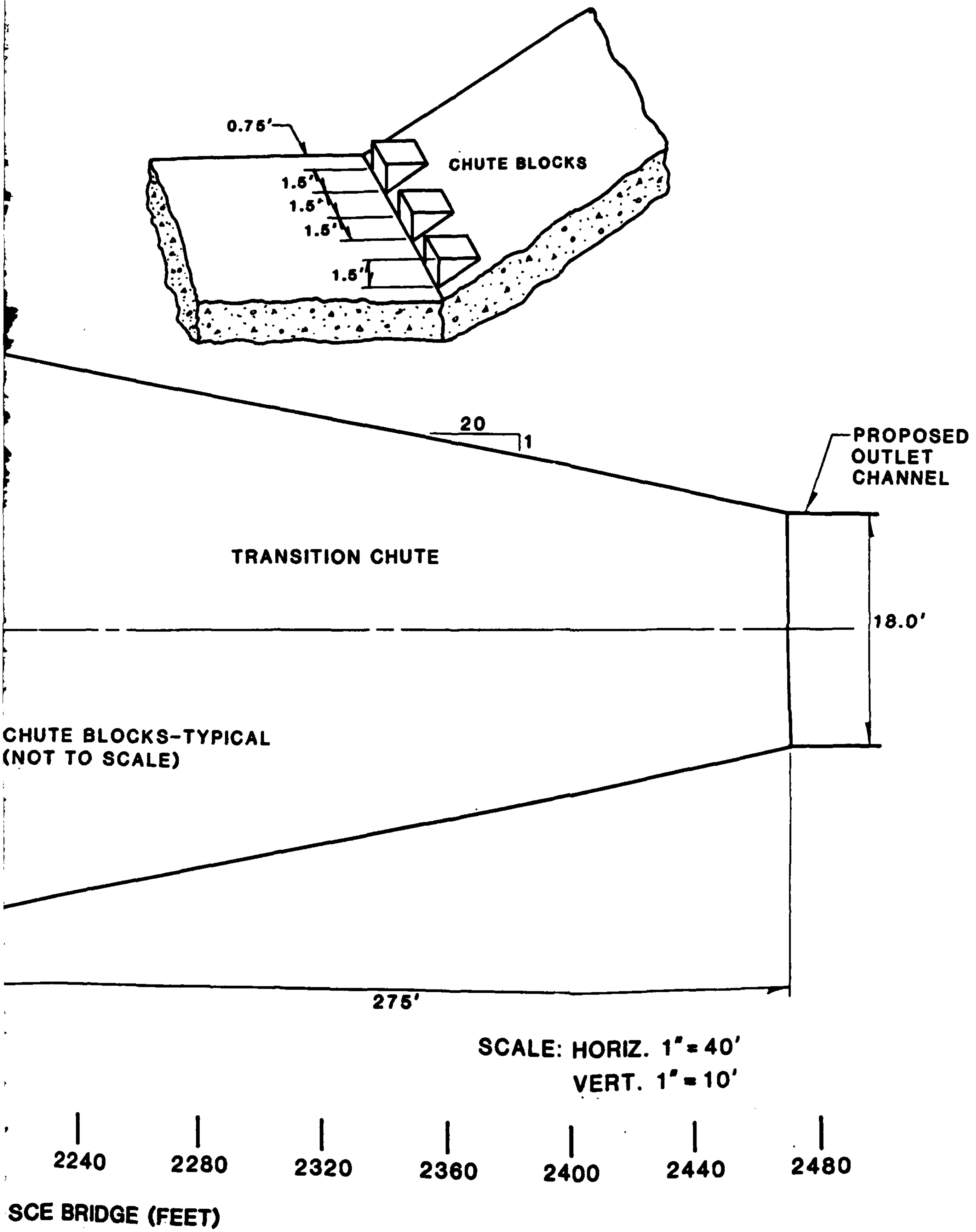
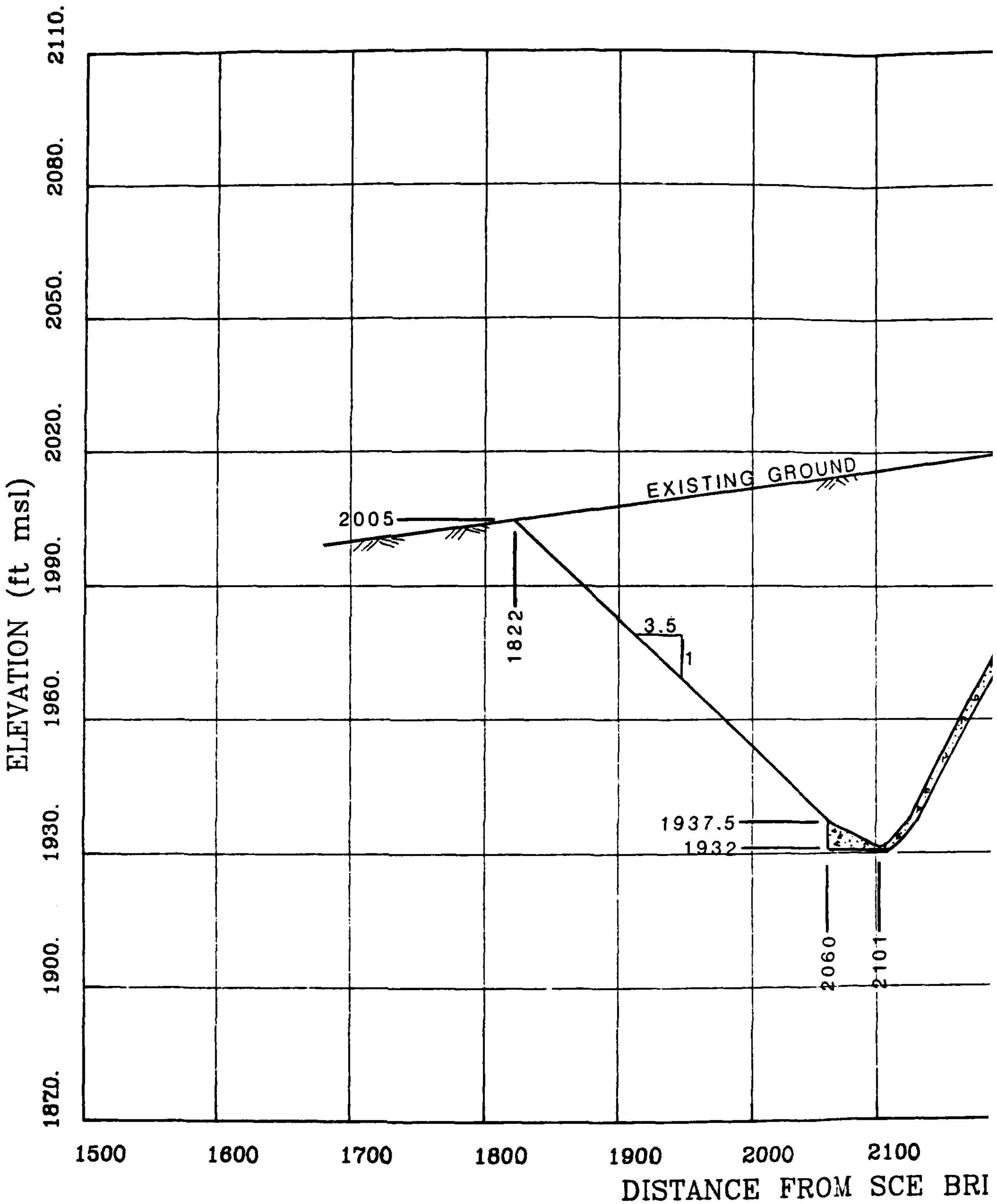


Figure 4.2. Plan view and detail of stilling basin.



ELEVATION (ft msl)
2110.
2080.
2050.
2020.
1990.
1960.
1930.
1900.
1870.
EXISTING GROUND
2005
1822
3.5
1
1937.5
1932
2060
2101
1500
1600
1700
1800
1900
2000
2100
DISTANCE FROM SCE BRI

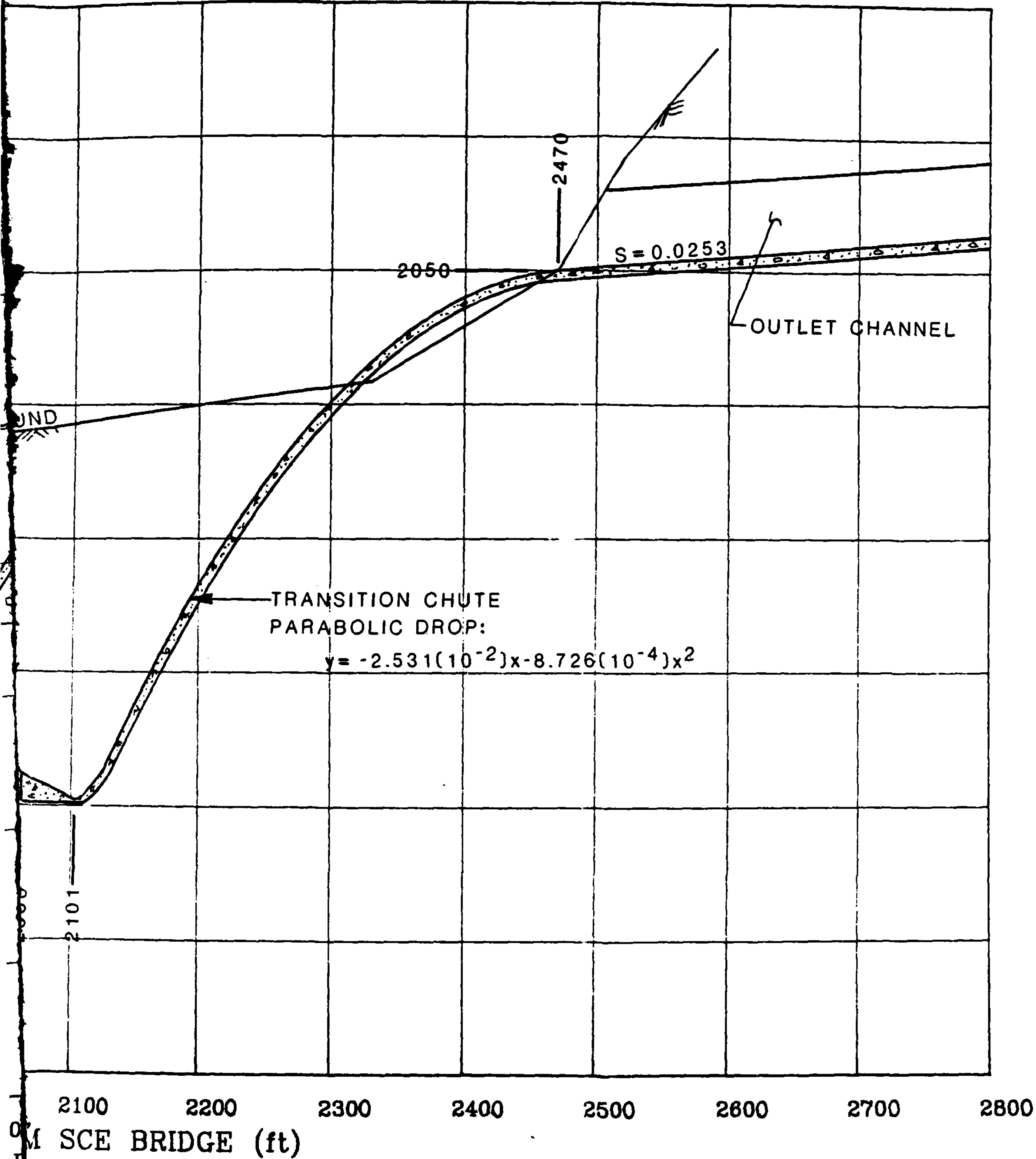


Figure 4.3. Profile of submerged slotted bucket.

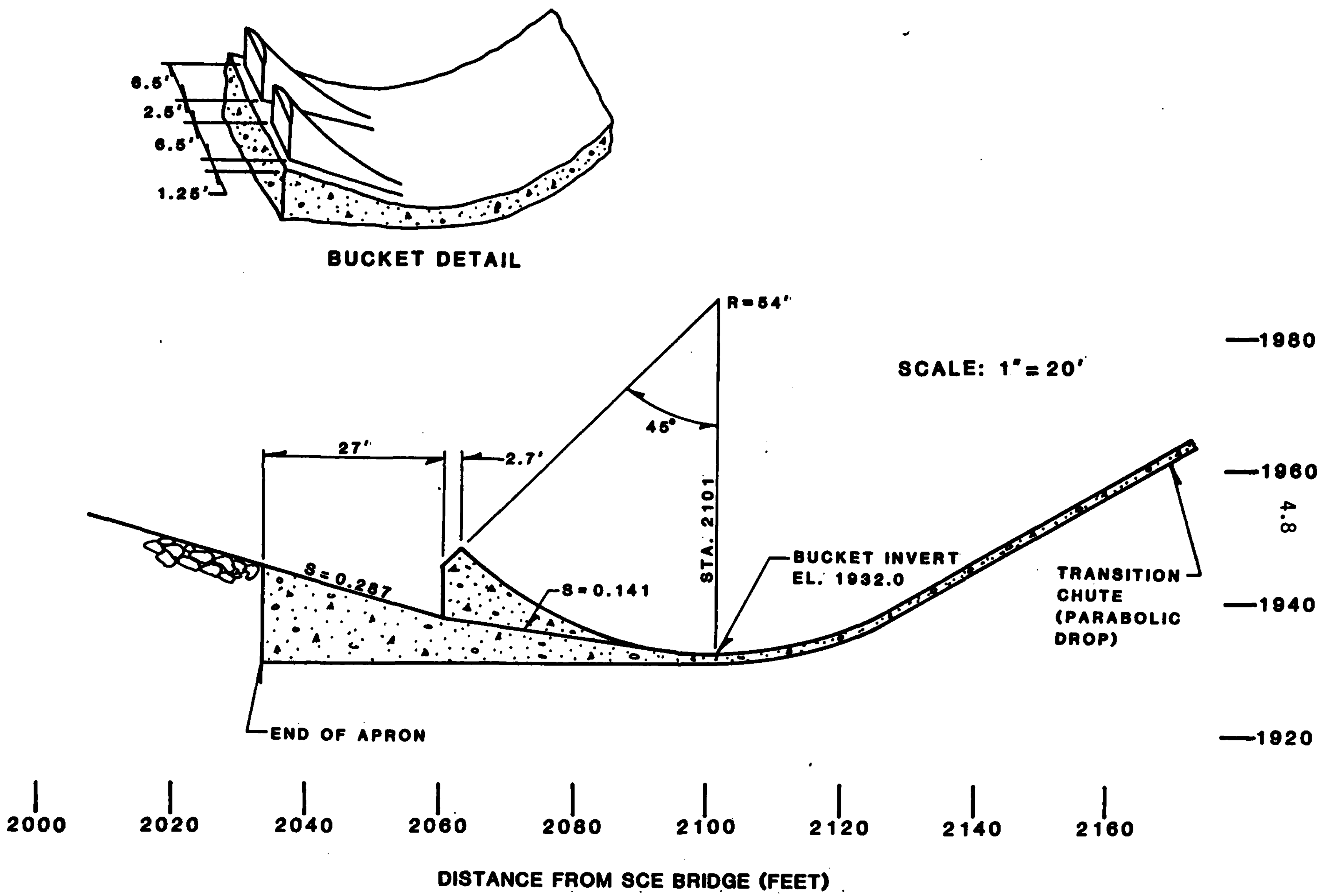


Figure 4.4. Detail of submerged slotted bucket.

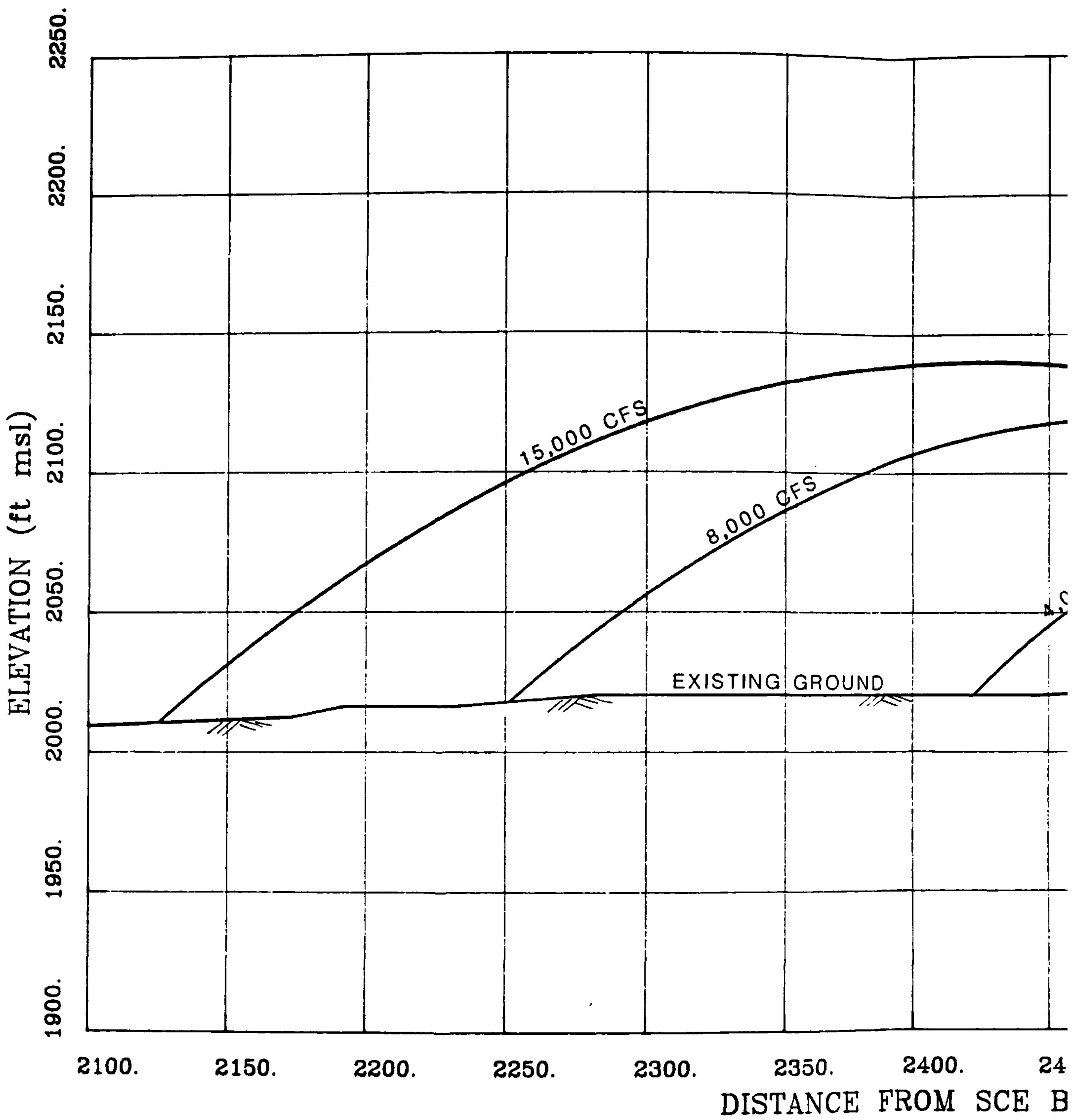
tively. The cost of constructing a concrete chute for the bucket and excavating at 3:1 sideslopes to provide stable sidewalls would be prohibitive. As with the stilling basin, the submerged bucket is not a preferred method for dissipating energy of very high velocity flows.

4.2.6 Alternative 6: Flip Bucket

The flip bucket alternative would require only the construction of the flip bucket (a relatively minor structural concrete addition) at the end of the outlet portal exit channel. This alternative would result in the formation of a natural scour hole much like the do nothing alternative. The plan view of this alternative is shown on Sheet 6 and the profile and detail sketches are given in Figures 4.5 and 4.6, respectively. However, the scour hole associated with the flip bucket would be located farther away from the outlet portal exit channel than the Alternative 1 scour hole. Also, because there is energy dissipation associated with the flow trajectory imparted by the flip bucket, the flip bucket scour hole would be shallower than the Alternative 1 scour hole for any discharge. However, adequate energy dissipation would not be realized until a natural scour hole is formed, the material eroded during formation of the scour hole would be deposited in the channel, and the outlet portal exit channel would not be protected from scour action caused by low flows.

4.2.7 Alternative 7: Flip Bucket with Preformed Plunge Pool and Toe Protection for the Outlet Portal Exit Channel

Including a preformed plunge pool and toe protection for the outlet portal exit channel with the flip bucket alternative would provide energy dissipation with the preformed plunge pool, provide protection for the outlet portal exit channel with a concrete cutoff wall at the bucket, and eliminate most of the deposition associated with natural formation of the scour hole. Excavation of the plunge pool would be relatively extensive, but the low cost of constructing the flip bucket make this a cost-effective solution. Estimation of the potential depth of scour and the scour hole configuration are explained in Appendix E.



ELEVATION (ft msl)
2250.
2200.
2150.
2100.
2050.
2000.
1950.
1900.
15,000 CFS
8,000 CFS
EXISTING GROUND
2100.
2150.
2200.
2250.
2300.
2350.
2400.
24
DISTANCE FROM SCE B

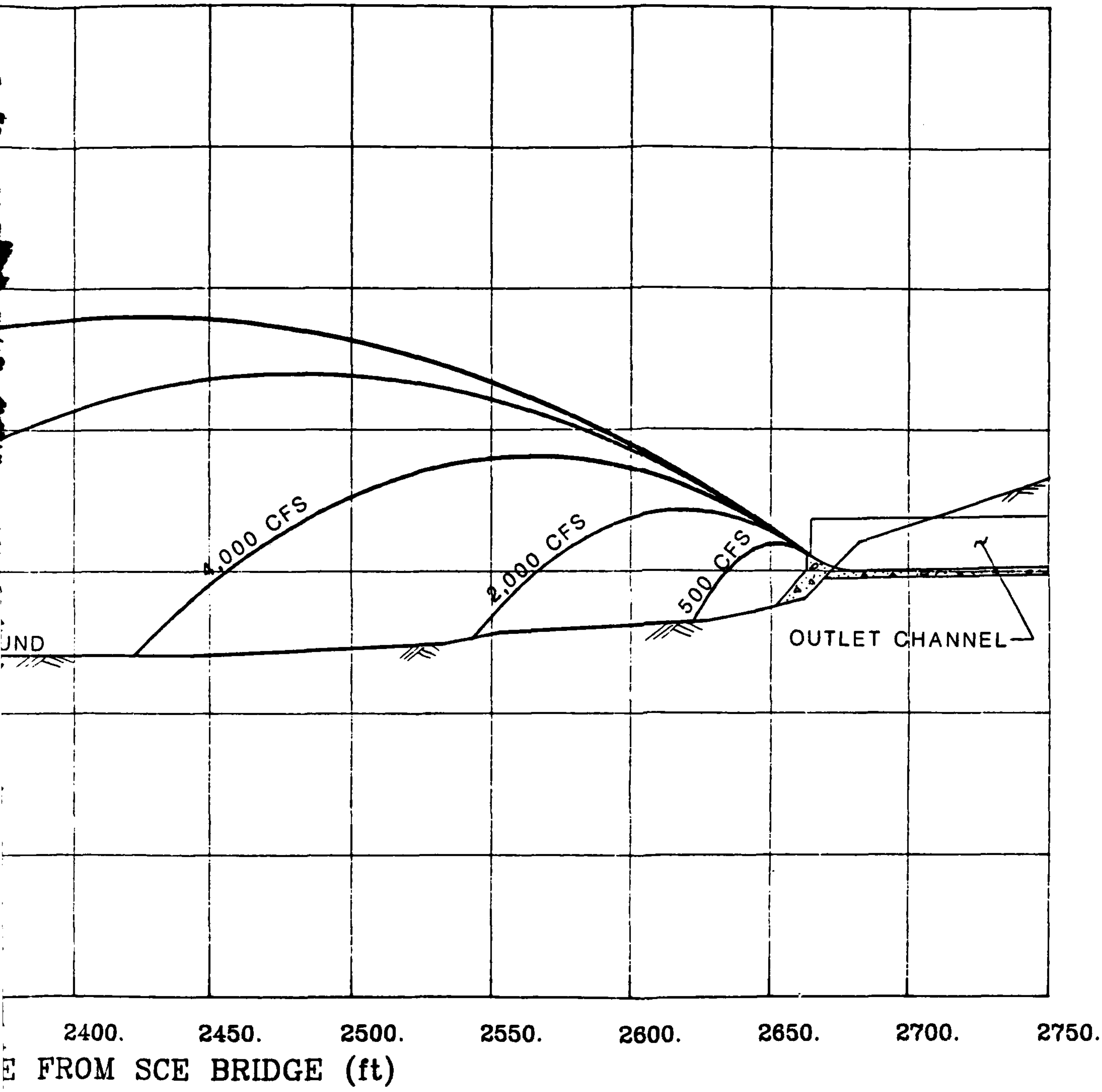


Figure 4.5. Profile of the flip bucket alternative and jet trajectories.

4.2.8 Alternative 8: Extend Toe Protection for Proposed Dam Embankment to Bedrock

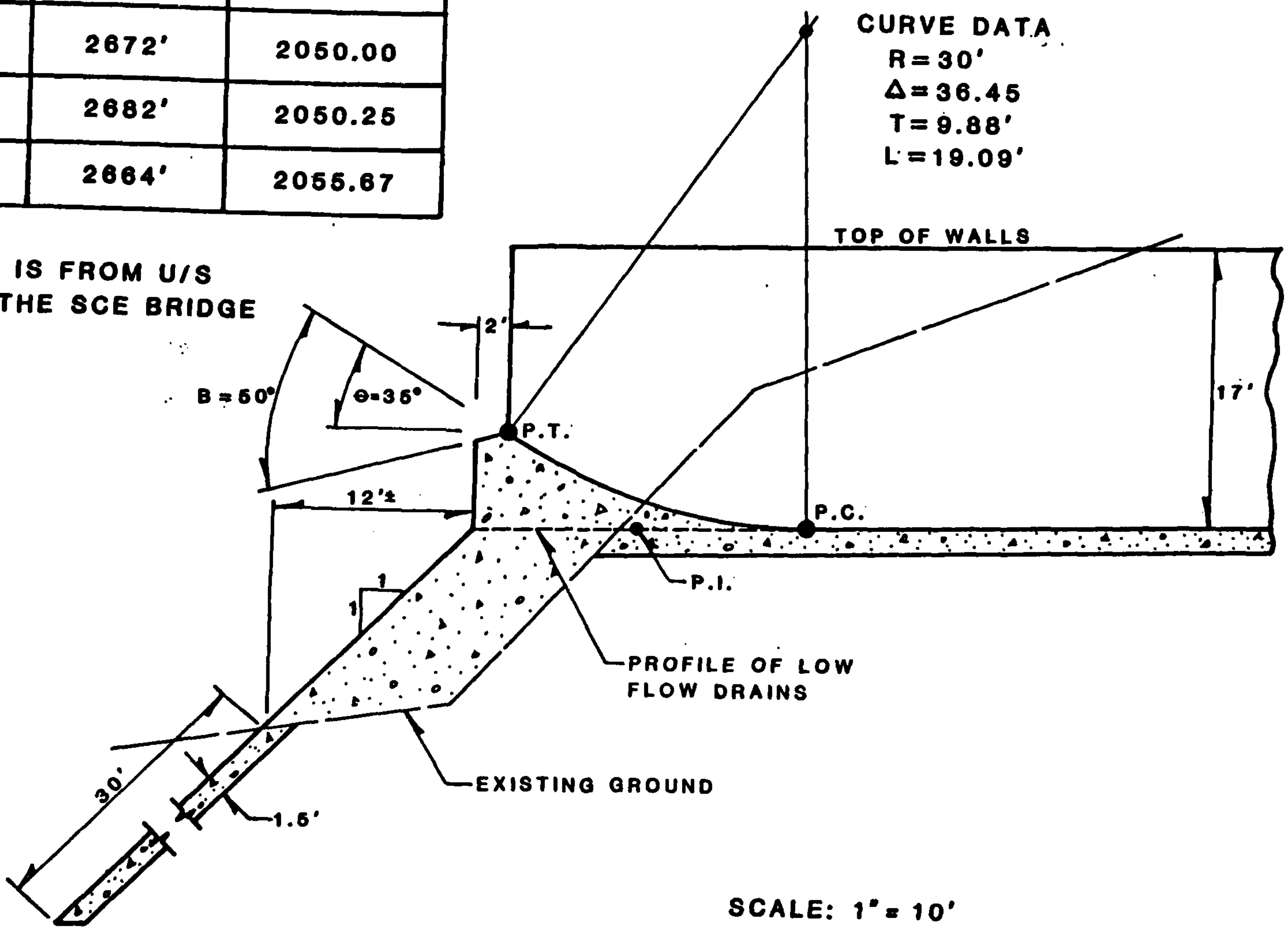
Extending toe protection for the proposed dam embankment to bedrock was investigated as a method to insure the stability of the earth-filled dam against downstream scour and subsequent headcutting. This would be a costly alternative because bedrock is located approximately 100 feet below existing ground in the vicinity of the embankment and the worst-case scour depth for all alternatives is less than 50 feet. The extended toe protection would not provide any real benefits because the dam embankment would not be threatened regardless of which alternative is selected. Headcutting in the existing channel would not occur since, for postproject conditions, there would not be flows in the channel directly downstream of the dam. In addition, the worst-case scour hole, that which would be formed for Alternative 1, would meet existing ground at least 350 feet downstream of the embankment.

4.2.9 Alternative 9: Channelization Upstream of Greenspot Road

Channelization upstream of Greenspot Road was considered in order to reduce flooding, stabilize the channel bed, and eliminate slope failure of the canyon walls. An earth-leveed channel with a 75-foot bottom width was considered upstream of the SCE bridge. The channel would be approximately 2,000 feet long and carry flows from the recommended energy dissipating measure to the SCE bridge. The levees would be up to 10 feet high and provide a minimum of 3 feet of freeboard for 8,000 cfs. The channel would be capable of carrying 15,000 cfs, but would provide only minimal freeboard. However, even without channelization postproject flooding would be considerably reduced as compared to preproject flooding. Upstream of the Mill Creek confluence, hydrology provided by the COE indicates 72,700 cubic feet per second (cfs) as the preproject, standard project flood (SPF) peak discharge, while the maximum normal operating release for postproject conditions would be only 8,000 cfs. (This discharge is lower than the preproject 10-year flow. The postproject outlet capacity release of 15,000 cfs is less than the preproject 25-year flow.) Therefore, the extent of flooding would be reduced by constructing the dam and outlet facilities as they are currently designed. The estimated flood plain boundaries (using existing topography as a fixed bed) for the preproject SPF and postproject discharges of 8,000 and 15,000 cfs are shown on Sheet 2 for comparative purposes. A 1980 photorevision to the USGS quadrangle map indicates a levee extending upstream from Greenspot Road for a distance of

POINT	DISTANCE*	ELEVATION
P.I.	2672'	2050.00
P.C.	2682'	2050.25
P.T.	2664'	2055.67

*DISTANCE IS FROM U/S FACE OF THE SCE BRIDGE



SCALE: 1" = 10'

Figure 4.6. Detail of flip bucket with toe protection.

approximately 1,500 feet along the west side of the channel. Based on field observations and the information given on the quadrangle map, the levee would not significantly alter the flood plain boundaries shown on Sheet 2 because flows could possibly escape behind the levee at its upstream end.

The existing channel does not require stabilization because it was found that even for the maximum postproject discharge of 15,000 cfs the channel would undergo only limited degradation. This is due to armoring by relatively high flows experienced in the past (see Chapter III for details). Finally, due to reduced flooding and a relatively stable channel bed, the potential for canyon wall slope failure would be reduced under postproject conditions. It should be noted that flows in excess of 15,000 cfs were recorded upstream of the SCE bridge in 1966 and 1969. Therefore, it is apparent that the bridge is structurally capable of withstanding floods of this magnitude. However, channelization upstream of the SCE bridge would allow flows less than or equal to the 15,000 cfs outlet capacity discharge to pass the bridge rather than overtopping the access road as is currently the case. The channelization configuration considered to enable the SCE bridge to pass 15,000 cfs is shown on Sheet 7.

4.2.10 Alternative 10: Drop Structures

Placing drop structures between the dam site and Greenspot Road was considered as an alternative measure for stabilizing the channel bed. Since, as indicated above, the existing channel is essentially stable for the entire range of postproject discharges due to armoring, it is felt that the drop structures are not necessary.

4.2.11 Alternative 11: Site-Specific Erosion/Flood-Control Structures

Protection of existing structures or facilities endangered by inundation or scour with site-specific erosion/flood-control protection measures was considered. To determine whether or not these measures would be necessary, the relative impacts of the preproject, SPF discharge (72,700 cfs) and the postproject, maximum normal operating discharge (8,000 cfs) on the various existing structures and facilities, and necessary protection measures were investigated. They are as follows (see Sheet 2 for locations of facilities).

1. SAR #3 Powerhouse is not affected by either pre- or postproject flows.

2. SCE Flume passes both pre- and postproject flows. Scour potential is reduced for postproject conditions.

3. SCE Bridge cannot pass either the pre- or postproject discharges. However, it has demonstrated the structural ability to withstand 15,000 cfs flows (in 1966 and 1969). Further, scour potential is reduced for postproject conditions.

4. Other SCE Buildings and Power Poles - Two buildings and one power pole are affected by preproject flows. None of these structures are impacted by postproject flows.

5. San Bernardino County Water Conservation District (SBCWCD) Canal Headworks could possibly sustain damage due to scour occurring behind the structure for preproject flows. For postproject conditions, all flows are contained in front of the structure where scour potential is minimized due to ponding caused by the SCE bridge.

6. SBCWCD Cobble Weir, at the SCE bridge, is subject to scour action at its downstream end by both pre- and postproject flows. The scour potential is less for postproject than for preproject conditions.

7. Greenspot Pipeline is buried in the Santa Ana River channel along the left bank. The pipeline may be subject to damage due to scour under preproject conditions since the SPF inundates much of the area that covers the pipeline. However, only a small portion of the pipeline is located in the postproject flood plain. Along this short reach, the pipeline has at least 10 feet of cover. Since the potential postproject scour depth for this reach is less than 1.0 foot, the pipeline will not be impacted by normal operation postproject flows. (See Chapter V for discussion of emergency releases).

8. Redlands Aqueduct does not lie in either the preproject or normal operation postproject flood plains. (See Chapter V for discussion of emergency releases).

9. Morton Canyon Connector is partially located in the preproject flood plain, but is not located in the postproject flood plain.

10. Bear Valley Highline is not impacted by either preproject or normal operation postproject flows. (See Chapter V for discussion of emergency releases).

11. Redlands Tunnel is located entirely in the preproject flood plain. Only a small portion of the upstream end of the tunnel lies in the postproject flood plain. The only profile available for the tunnel is a 1905 profile which uses an unknown datum. However, this profile indicates that the portion of the tunnel in the postproject flood plain was located a minimum of 50 feet below existing ground in 1905. Based on this information and the fact that the potential impact to the tunnel for postproject conditions is less than for preproject conditions, it is felt that protection is not required.

12. Greenspot Road Bridge appears to be hydraulically capable of passing both pre- and postproject flows. Scour potential is reduced for postproject conditions. However, the 1969 flood which had a peak discharge of approximately 15,000 cfs at Greenspot Road knocked the bridge from its foundation. Although the bridge was resurrected, it may not be structurally capable of withstanding 8,000 or 15,000 cfs flows, but flooding and therefore potential for structural failure is reduced for postproject conditions.

Based on the above information it is evident that postproject flows either do not impact the structures and facilities in the Santa Ana Canyon or show reduced impacts as compared to preproject conditions.

4.2.12 Alternative 12: Minimizing the Impact to the Endangered Species

Minimizing the impact to the endangered species may require maintaining existing flooding patterns to the extent possible. From Figures 4.7 and 4.8 it can be seen that, if channelization is not employed, both the maximum pre- and postproject flows inundate most of the area containing the endangered species (Eriastrum). It is also evident that both the 2-year preproject and 5-year postproject flood plains encompass approximately one-half of the endangered species area, while the 2-year postproject flood impacts approximately one-quarter of the endangered species area. Therefore, it can be concluded that while the project would reduce the frequency and extent of flooding of the endangered species, this reduction will not be substantial and the Eriastrum would still be subject to flooding on a regular basis.

4.3 Preliminary Evaluation of Conceptual Alternative Components

All but 3 of the 12 alternatives were eliminated for not meeting the performance criteria established in Section 4.1.2 or being less economically feasible than a competing alternative. A detailed analysis of the three remaining alternatives will be conducted in a later section.

Four alternatives were eliminated due to their ineffectiveness; that is, the adverse impacts they were intended to minimize would not be realized in the study area under postproject conditions. These particular adverse impacts are increased flooding and degradation. Since postproject releases would be substantially smaller than preproject discharges, flooding would be correspondingly reduced. Although the dam would eliminate the upstream sediment supply to the study reach, only limited degradation would take place. This is due to the projection that the reach upstream of Greenspot Road will be heavily

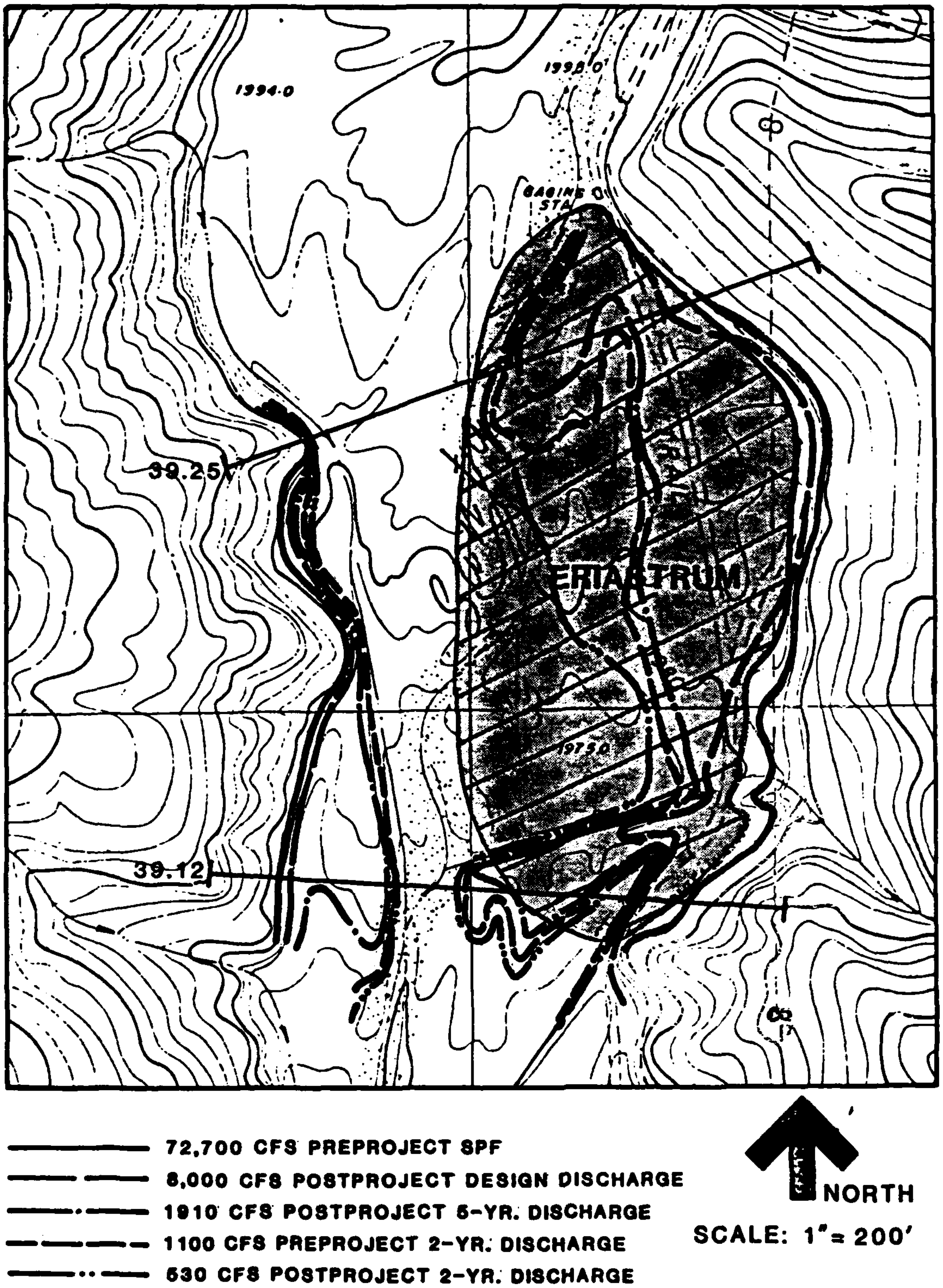


Figure 4.7. Plan view of the postproject impact to the endangered species.

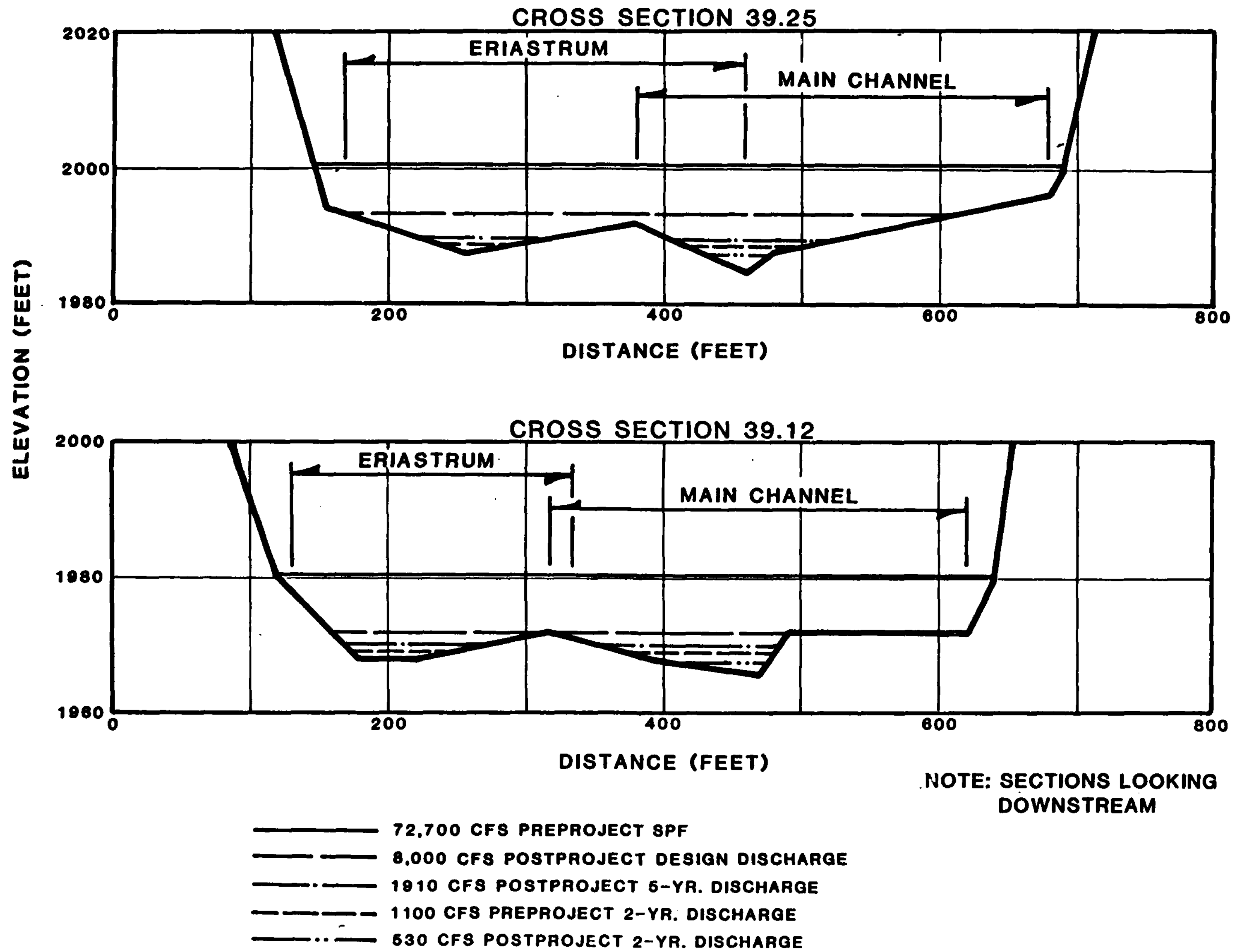


Figure 4.8. Section view of the postproject impact to the endangered species.

armored by postproject flows. The alternatives eliminated due to ineffectiveness are:

1. Alternative 4, stilling basin with channel stabilizer,
2. Alternative 9, channelization upstream of Greenspot Road,
3. Alternative 10, drop structures, and
4. Alternative 11, site-specific erosion/flood-control protection.

It should be noted that the channelization alternative would completely eliminate flooding of the Eriastrum.

Extending toe protection for the dam embankment to bedrock was determined to be an unnecessary measure. The embankment would not be threatened by postproject releases. The worst-case scour hole would remain at least 350 feet from the embankment. Furthermore, only minor runoff from the embankment itself and the canyon walls would feed the channel downstream of the dam embankment. Due to the small flows between the embankment and the scour hole, and the significant armoring potential of the channel, headcutting would be minimal.

Both Alternative 1 and the flip bucket alternative were eliminated for the following reasons:

a. Inability to dissipate energy prior to the formation of a scour hole,

b. Deposition of nearly 55,000 and 100,000 cubic yards of material, respectively, associated with the formation of the scour hole, and

c. High potential for damaging the outlet channel by slope failure due to scour action caused by low flows.

Preliminary cost estimates were made for those alternative components which were considered to be reasonably viable. These estimated costs are given in Table 4.1 and include 20 percent for contingencies. Summaries of the cost estimates are provided in Appendix F. The stilling basin and submerged slotted bucket were eliminated due to their uncertain technical feasibility for design conditions, primarily related to the lack of significant tailwater depth and relatively high costs.

4.4 Final Evaluation of Conceptual Alternatives

Completion of the preliminary evaluation left the following three conceptual alternatives to be given more detailed consideration:

Table 4.1. Preliminary Cost Estimates for Energy Dissipating Conceptual Alternatives.

Alternative Description	Cost* (Thousands of Dollars)
Alternative 2: Preformed plunge pool and toe protection for the outlet portal exit channel	315**
Alternative 3: Stilling basin	870
Alternative 5: Submerged slotted bucket	867
Alternative 7: Flip bucket with preformed plunge pool and toe protection for the outlet portal exit channel	464**

* Includes 20 percent contingency.
** Based on detailed analysis described in Section 4.4; does not include the cost of anchoring the cutoff wall.

1. Alternative 2, preformed plunge pool and toe protection for outlet portal exit channel,
2. Alternative 7, flip bucket with preformed plunge pool and toe protection for outlet portal exit channel, and
3. Alternative 12, minimizing the impact to the endangered species.

Alternative 2 and the flip bucket alternative are truly alternatives in that they represent two different energy dissipating measures. The third alternative is actually a separate component which can be implemented in conjunction with either of the two energy dissipating measures.

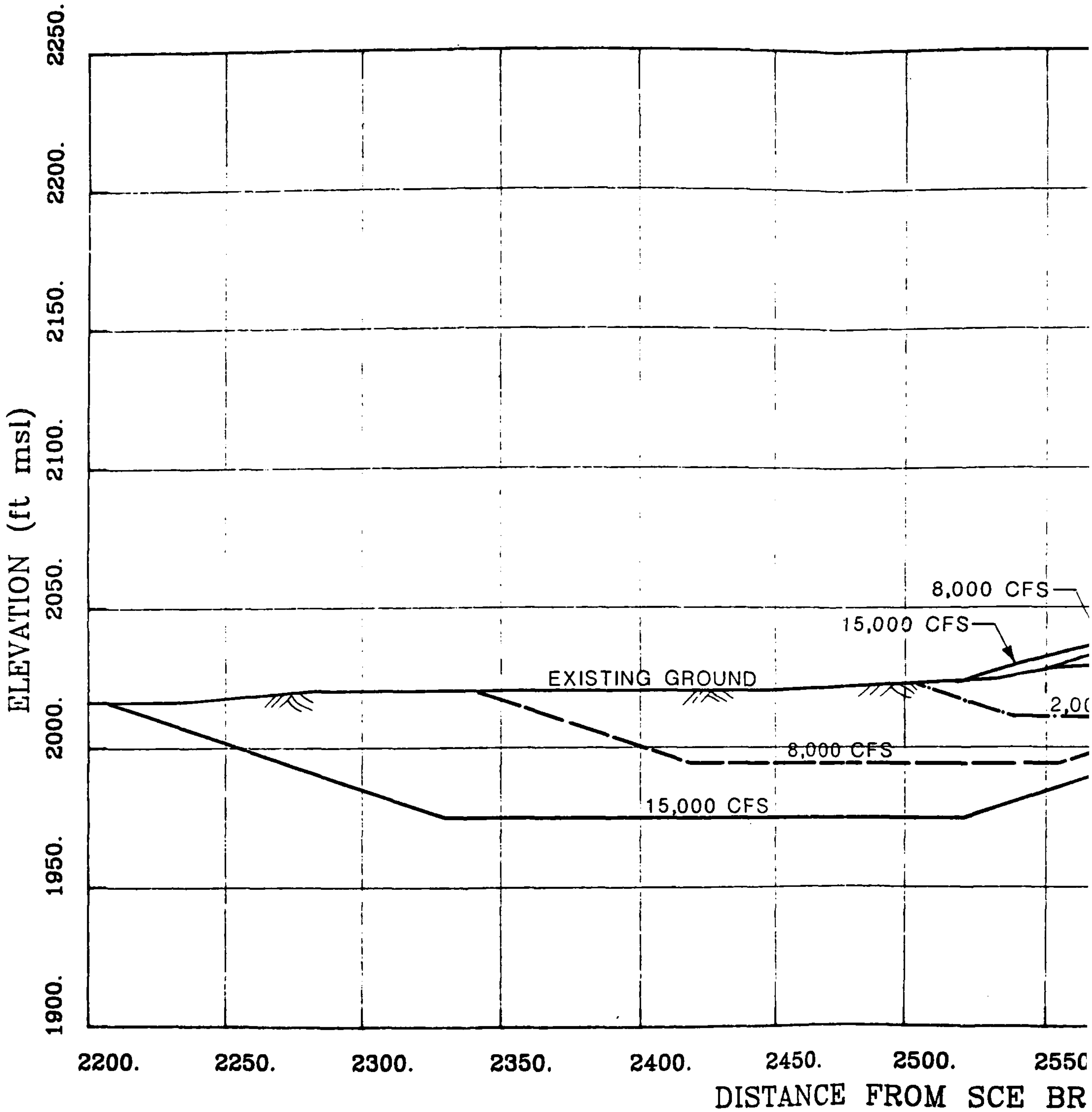
4.4.1 Alternative 2: Preformed Plunge Pool and Toe Protection for Outlet Portal Exit Channel

The potential dimensions for the scour holes associated with the 500, 2,000, 8,000, and 15,000 cfs releases were calculated as explained in Appendix E. The 500, 2,000, and 8,000 cfs scour depths were computed to determine the size and location of the composite scour hole resulting from normal operating releases. The 15,000 cfs scour depth was calculated to determine the extent of the scour hole developed under maximum conduit capacity conditions. The scour depths for these three discharges are given in Table 4.2. The profiles of these four scour holes are shown in Figure 4.9. The plan view of the combined scour hole for 2,000 and 8,000 cfs, and the extent of scour for 15,000 cfs are shown on Sheet 3.

It was planned that the preformed plunge pool would be excavated to approximately two-thirds of the maximum depth for 8,000 cfs. For this condition, preforming the scour hole would require excavating approximately 43,000 cubic yards of material. It is recommended that the coarse excavated material (diameter $\geq$ 6 inches) be retained and used to line the plunge pool once excavation is complete. This would result in a more stable plunge pool and possibly reduce the ultimate scour depth. The size of cutoff wall required to protect the outlet was found to be controlled by the potential scour hole associated with the 2,000 cfs release. This can be seen from Figure 4.9 and Sheet 3. Due to the large scour depth at the outlet, it was assumed that the cutoff wall should be placed at a 1:1 slope. Extending the cutoff wall 10 vertical feet below and 10 lateral feet beyond the scour hole (at the end of the outlet portal exit channel) and including 15-foot tie-back walls, were the criteria used for designing the cutoff wall. The resulting

Table 4.2. Best Estimate Scour Depths for Alternative 2, Preformed Plunge Pool and Toe Protection for Outlet Portal Exit Channel.

Discharge (cfs)	Best Estimate Scour Depth (ft)
500	10
2,000	20
8,000	34
15,000	48



ELEVATION (ft msl)
2250.
2200.
2150.
2100.
2050.
2000.
1950.
1900.
8,000 CFS
15,000 CFS
EXISTING GROUND
2,00
8,000 CFS
15,000 CFS
2200.
2250.
2300.
2350.
2400.
2450.
2500.
2550
DISTANCE FROM SCE BR

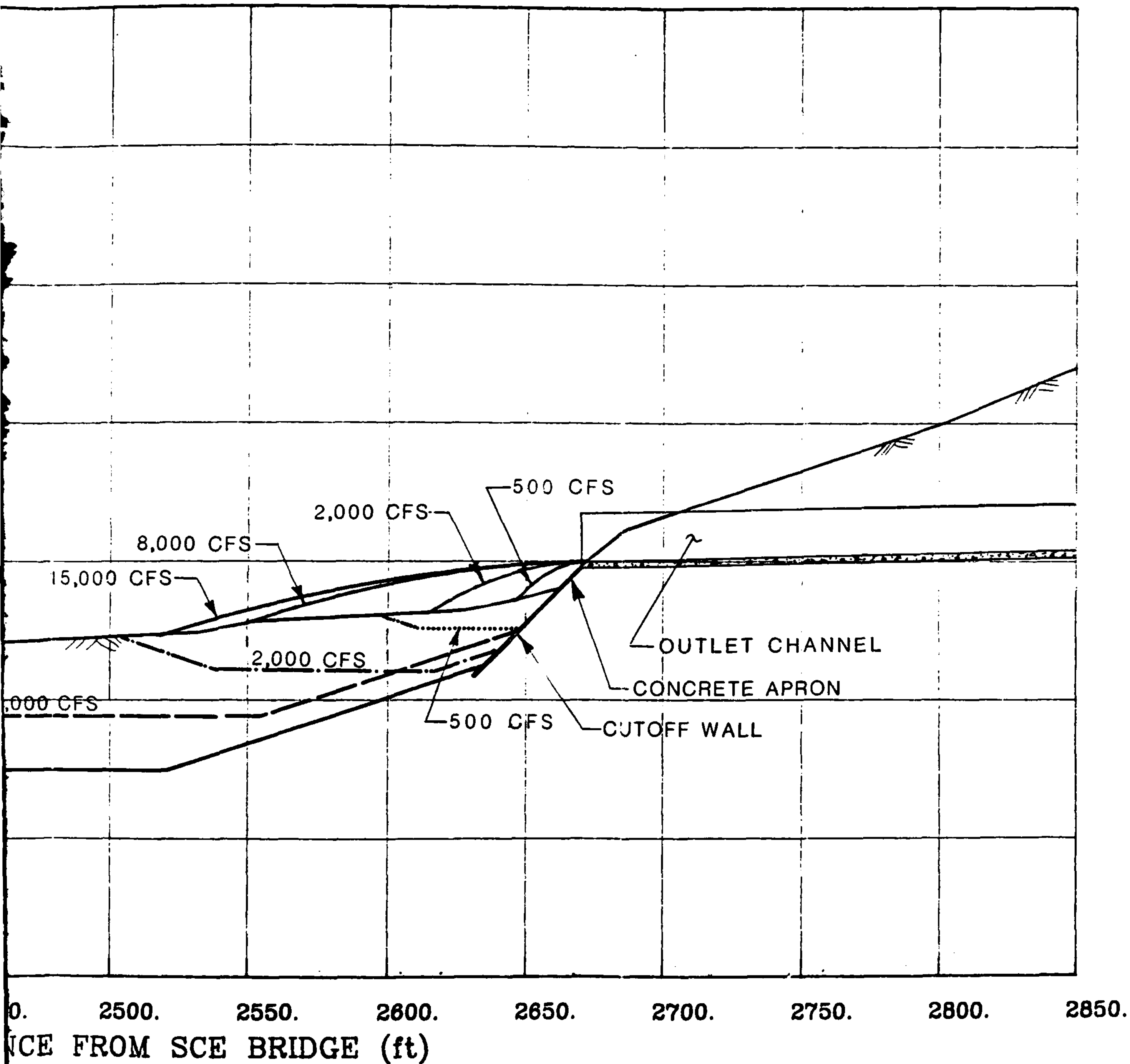


Figure 4.9. Profile of scour holes and Alternative 2.

cutoff wall includes a 60-foot-long sloping face on a 1:1 slope (42 vertical feet) and 218-foot top length, as shown on Sheet 3 and Figure 4.9. The estimated cost for this alternative, given in Table 4.1, does not include the cost of anchors for the cutoff walls, but assumes a conservative cutoff wall thickness of 1.5 feet. The cost estimate also does not include the expense of sorting the excavated material and lining the plunge pool.

The following are the advantages associated with Alternative 2, preformed plunge pool and toe protection for outlet portal exit channel:

1. It would be more economical than the flip bucket alternative,
2. It would require less excavation than the flip bucket alternative,
3. Siltation of the scour hole should not be a significant problem,
4. It would be effective for all normal operating discharges,
5. Abrasion to concrete during sluicing operations, if any, would be minimal, and
6. Downstream impact would be acceptable.

The disadvantages associated with Alternative 2, preformed plunge pool and toe protection for outlet portal exit channel are:

1. The scour hole would be located very close to the outlet portal exit channel,

2. Energy dissipation would not be as effective as for the flip bucket alternative due to flow trajectory,

3. The scour hole would be slightly deeper than the flip bucket alternative (34 feet versus 26 feet at 8,000 cfs),

4. Installation of the cutoff wall for toe protection of the outlet portal exit channel would be more difficult than for the flip bucket alternative, and

5. It is difficult to accurately predict the actual trajectory of flow and size of the scour hole. Physical modeling must be carefully performed to verify calculations.

4.4.2 Alternative 7: Flip Bucket Alternative (With Preformed Plunge Pool and Toe Protection for Outlet Channel)

The potential dimensions for the scour holes associated with the 500, 2,000, 4,000, 8,000 and 15,000 cfs releases were calculated, as explained in Appendix E. The scour depths associated with these five discharges are given in Table 4.3. The 15,000 cfs scour depth was computed to determine the extent of scour for conduit capacity release conditions (worst case scenario). Scour

Table 4.3. Estimated Scour Depths for the Flip Bucket With Preformed Plunge Pool and Toe Protection for Outlet Channel Alternative.

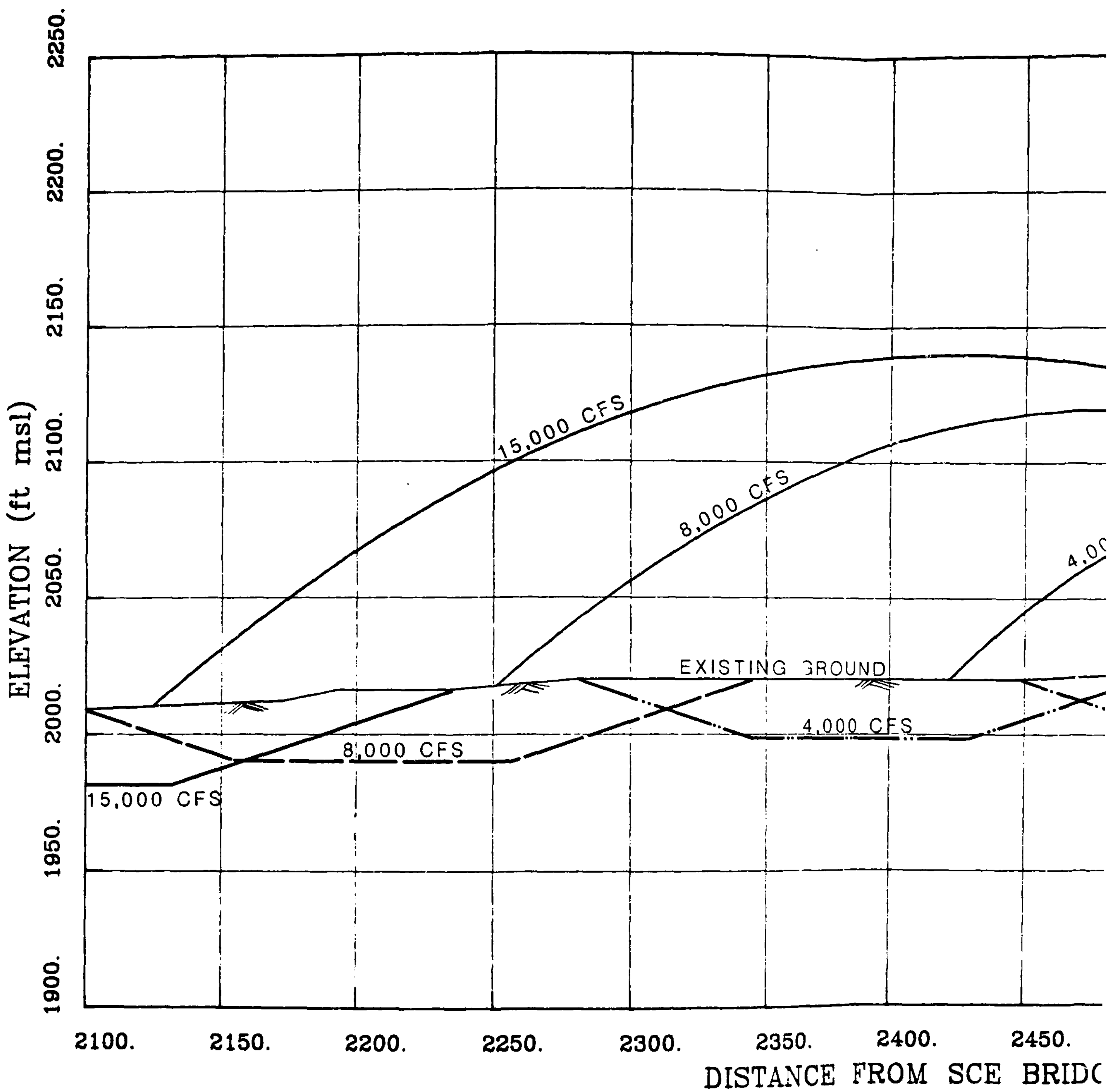
Discharge (cfs)	Estimated Scour Depth (ft)
500	9
2,000	17
4,000	21
8,000	26
15,000	28

depths for the discharges up to 8,000 cfs were calculated to determine the size and location of the scour hole associated with normal operation releases. The profiles of all five scour holes are shown in Figure 4.10. From this figure, it is evident the range of normal operation discharges will result in a long scour hole of continuously varying depth. Using the four normal operation scour holes, the composite scour hole associated with all discharges less than 8,000 cfs was developed. This scour hole and the extent of scour for 15,000 cfs are shown on Sheet 6.

It was again planned that the preformed plunge pool would be excavated to approximately two-thirds of maximum depth. For this condition, preforming the composite scour hole would require excavating approximately 77,000 cubic yards of natural material. It is again recommended that the coarse excavated material be used to line the plunge pool. The size of cutoff wall required to protect the outlet channel was found to be controlled by the scour hole associated with the 500 cfs release. This can be seen from Figure 4.10 and Sheet 4. It was again assumed that the cutoff wall should be placed at a 1:1 slope. Extending the cutoff wall 10 vertical feet below and 10 lateral feet beyond the scour hole (at the end of the flip bucket) and including 15-foot tie-back walls, were the criteria used for designing the cutoff wall. The resulting cutoff wall must have a 30-foot sloping face on a 1:1 slope, (22 vertical feet) and include a 200-foot top length, as shown on Sheet 6 and Figure 4.10. The estimated cost for this alternative, given in Table 4.1, does not include the cost of anchors for the cutoff wall, but assumed a conservative cutoff wall thickness of 1.5 feet. The cost estimate does not include the expense of sorting the excavated material and linig the plunge pool.

The following are the advantages associated with the flip bucket with preformed plunge pool and toe protection for outlet channel alternative:

1. The scour hole would be located farther away from the outlet portal exit channel than Alternative 2,
2. Slightly reduced scour depth (26 feet at 8,000 cfs) than for Alternative 2 (34 feet at 8,000 cfs),
3. It would be effective for all normal operating discharges,
4. There would be energy dissipation associated with dispersion and trajectory of the jet,
5. It requires a significantly smaller cutoff wall than the do nothing alternative,



2250.
2200.
2150.
2100.
2050.
2000.
1950.
1900.
ELEVATION (ft msl)
15,000 CFS
8,000 CFS
4,00
EXISTING GROUND
4,000 CFS
8,000 CFS
15,000 CFS
2100.
2150.
2200.
2250.
2300.
2350.
2400.
2450.
DISTANCE FROM SCE BRID

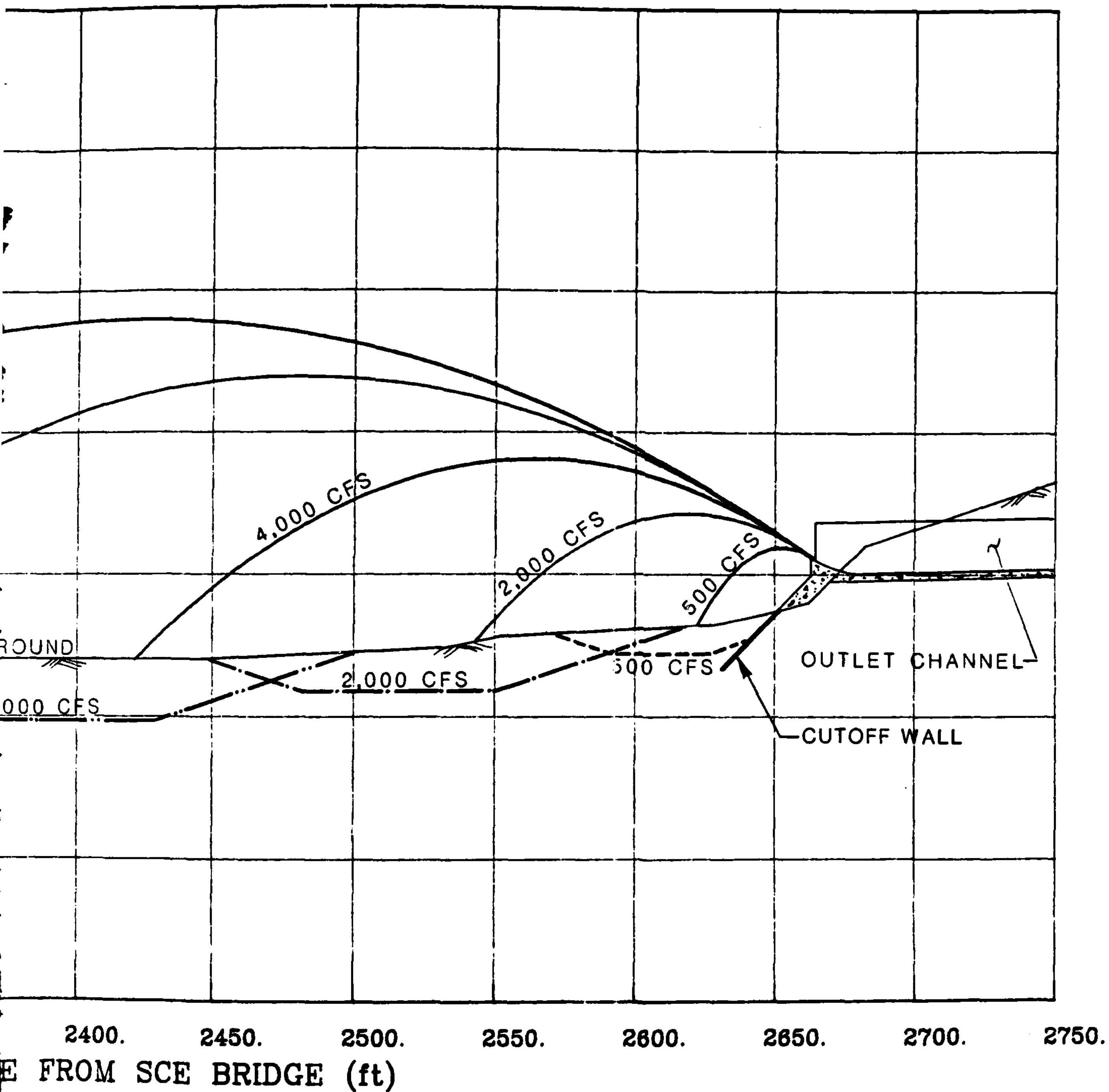


Figure 4.10. Profile of scour holes for the flip bucket alternative.

6. Siltation of the scour hole should not be a significant problem, and
7. Downstream impact would be minimal.

The disadvantages associated with the flip bucket with preformed plunge pool and toe protection for outlet channel alternative are:

1. It would be more expensive than Alternative 2 ($464,000 versus $315,000),
2. It requires excavation of a relatively large scour hole,
3. Abrasion of the flip bucket during sluicing operations, if any, may significantly reduce the design life of this alternative, and
4. It is difficult to accurately predict the actual trajectory of flow and size of the scour hole. Physical modeling must be carefully performed to verify calculations.

4.4.3 Minimizing Impact to Endangered Species

Since postproject conditions will reduce peak flows downstream of the dam site, it is inevitable that the endangered species will be impacted by the project. The result will be a reduction in flooding of the endangered species. As was indicated on Figures 4.7 and 4.8, the area of inundation of the endangered species for the 2-year return period, preproject condition would be about the same for the 5-year return period, postproject condition if channelization is not undertaken. On the other hand, if channelization measures are implemented, it is probable that flooding of the endangered species would cease completely.

4.5 Recommended Design Alternative

Based on the performance criteria developed for evaluating the alternative components, both Alternatives 2 and 7 are feasible energy dissipation measures. The least expensive of these two alternative components is Alternative 2, preformed plunge pool and toe protection for the outlet portal exit channel, and therefore is the recommended design alternative.

V. RECOMMENDED DESIGN ALTERNATIVE

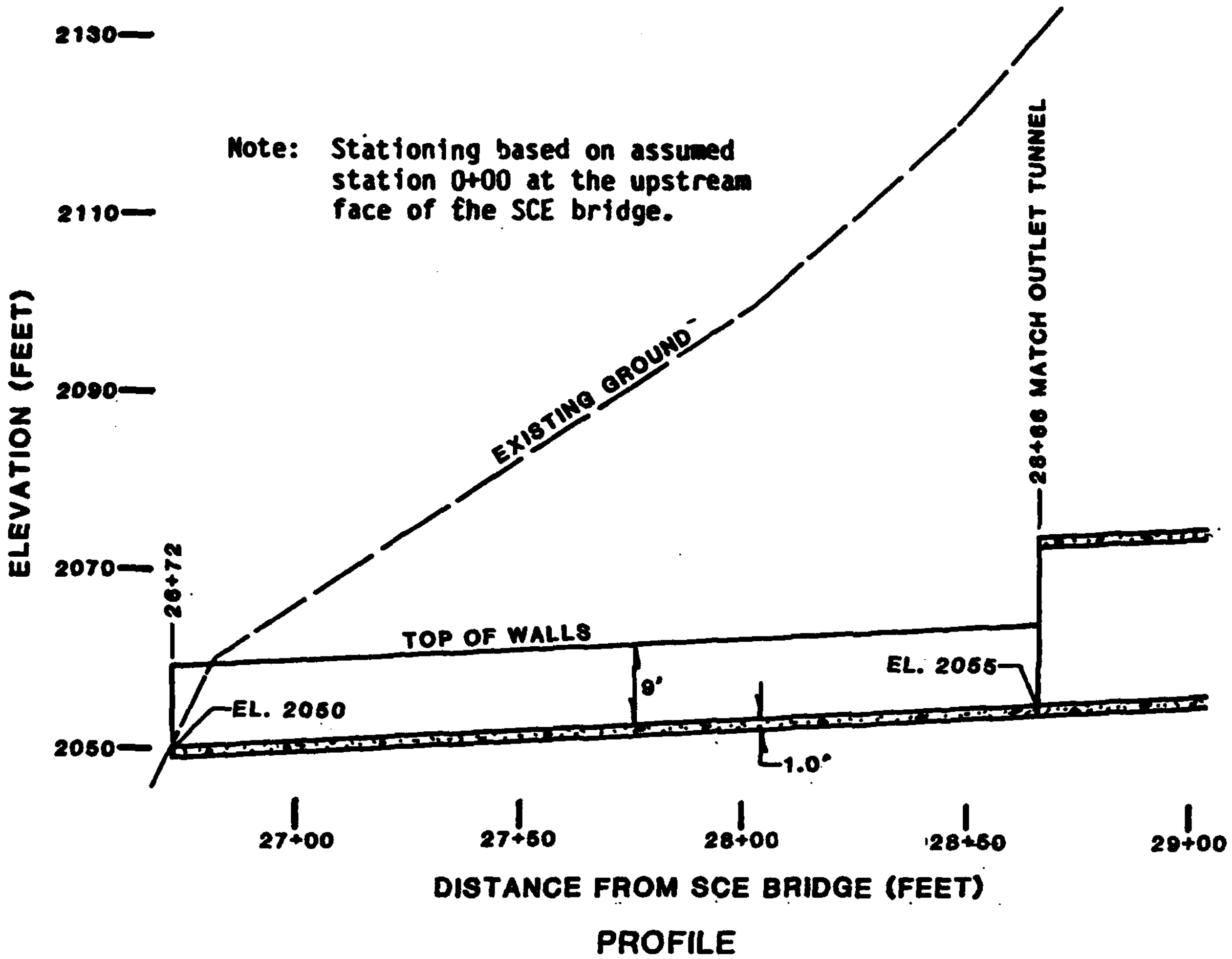
5.1 Alternative Description

The recommended design is Alternative 2 the preformed plunge pool and toe protection for the outlet channel. For this alternative, normal operation releases would discharge directly from the outlet portal exit channel. Under these conditions, a scour hole would be formed by the resulting water jet, eventually providing the required energy dissipation. By preforming a plunge pool downstream of the outlet portal exit channel, energy dissipation will be achieved for first-time releases. Therefore, it is recommended that a plunge pool be preformed to approximately two-thirds the ultimate scour depth. Without providing toe protection for the outlet portal exit channel, the channel would eventually be undercut by the scour hole induced by low flows. Therefore, it is further recommended that the existing ground below the outlet portal exit channel be protected with a concrete apron, and this apron be extended below ground at a 1:1 slope, to form a cutoff wall. This configuration is shown on Sheet 8. Some minor grading is required downstream of the plunge pool, as shown on Sheet 8, to provide a smooth transition to the existing channel.

The proposed outlet tunnel portal for Seven Oaks Dam is located approximately 194 feet upstream of the daylight line for the tunnel alignment (see Sheet 8). If the 18-foot-wide tunnel was continued at the current alignment and slope of 0.026, its invert would meet existing ground 194 feet southwest of the proposed portal at elevation 2,050 feet. Due to structural considerations, the tunnel cannot be extended to the daylight line. Therefore, an outlet portal exit was designed to connect the outlet tunnel to existing ground.

The high velocity (112 feet per second) associated with the design discharge of 8,000 cfs dictates that the outlet channel configuration match that of the proposed outlet tunnel. Therefore, the outlet portal exit channel must be 18 feet wide, on a slope of 0.026, and follow the outlet tunnel's alignment. The outlet channel design is shown in Figure 5.1. The 9-foot channel wall height will accommodate the design discharge assuming 50 percent bulking for air entrainment and provide 3 feet of freeboard. The channel alignment and areas of cut and fill are indicated on Sheet 8.

The stationing shown on Sheet 8 and on the figures in this chapter was adopted by SLA. It originates at the upstream face of the SCE bridge (Sta



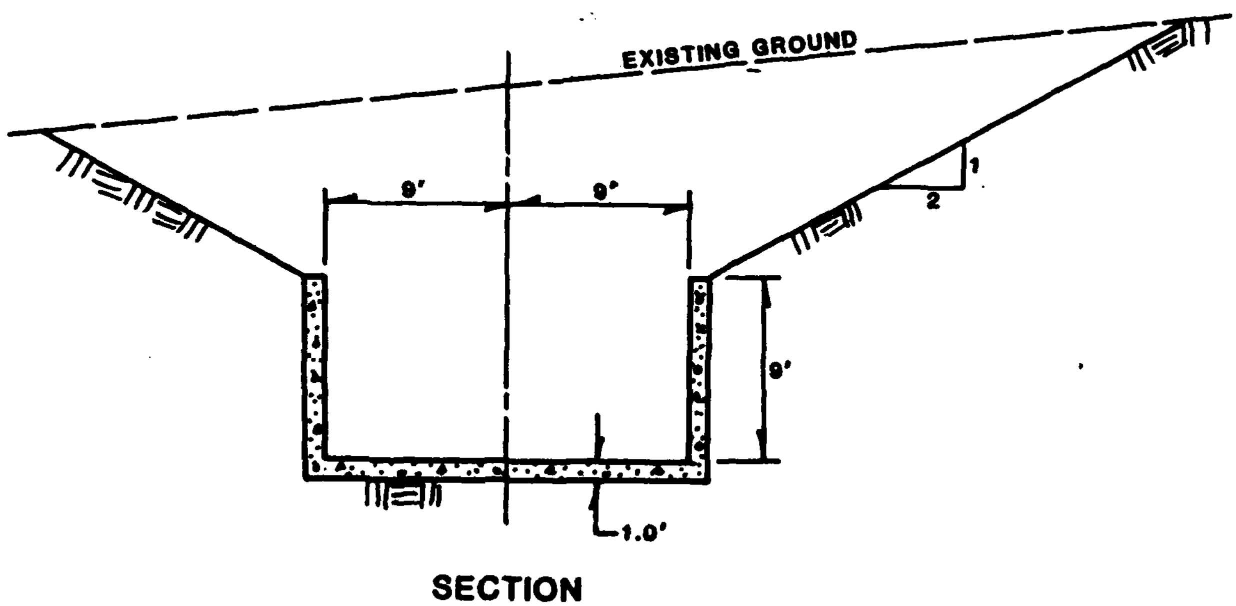


Figure 5.1. Profile and section view of the outlet channel.

0+00) and increases in the upstream direction. Subsequent to the completion of the preliminary design, the Portland District COE provided stationing which differs from the SLA stationing. Unlike the SLA stationing, the Portland District stationing increases in the downstream direction to accommodate the design of the outlet tunnel and inlet works. In order to transform between stationing systems, a match point was determined. The outlet tunnel portal is at SLA station 28+66, as shown on Sheet 8; this corresponds to Portland District station 28+46.

5.1.1 Scour Hole and Preformed Plunge Pool

Scour depths were computed for a range of discharges less than or equal to the design discharge (8,000 cfs), and the conduit capacity discharge (15,000 cfs), using the Chee and Yuen method (1985). For these calculations it was assumed that tailwater was negligible and the mean subsurface-particle diameter was 15 mm (the mean bed-material particle diameter was 90 mm). As explained in Appendix E, both best-estimate and maximum scour depths were determined. The Chee and Yuen method was used to compute the best-estimate depths. The maximum scour depths were conservatively computed by doubling the best-estimate depths. Details concerning these calculations are given in Appendix E. These scour depths are summarized in Table 5.1.

The Chee and Yuen method was felt to be the best method available for computing scour depths. Therefore, it is recommended that the alternative be designed for the best-estimate scour depths. However, it must be recognized that considerable uncertainty exists with any currently available method for computing scour depths. While maximum scour depths are provided as probable upper limits, it is essential that a model study be carefully conducted to determine actual scour depths and scour hole configurations before finalizing the design.

The typical scour hole configuration is described in Appendix E. The actual scour holes for 2,000 and 8,000 cfs are shown in Figures 5.2 and 5.3, respectively. The scour holes for all four discharges analyzed are shown superimposed on Figure 4.9. It is recommended that the preformed plunge pool be excavated to approximately elevation 2,005 feet. The depth of the resulting plunge pool will be approximately two-thirds of the potential scour hole depth. The preformed plunge pool is shown on Sheet 8 and in detail in Figure 5.4. It is suggested that when excavating the plunge pool, the coarse

Table 5.1. Summary of Best-Estimate and Maximum Scour Depths for the Recommended Design Alternative.

Discharge (cfs)	Scour Depth (ft) - Chee & Yuen	
	Best-Estimate	Maximum
500	10	20
2,000	20	40
8,000	34	68
15,000	48	96

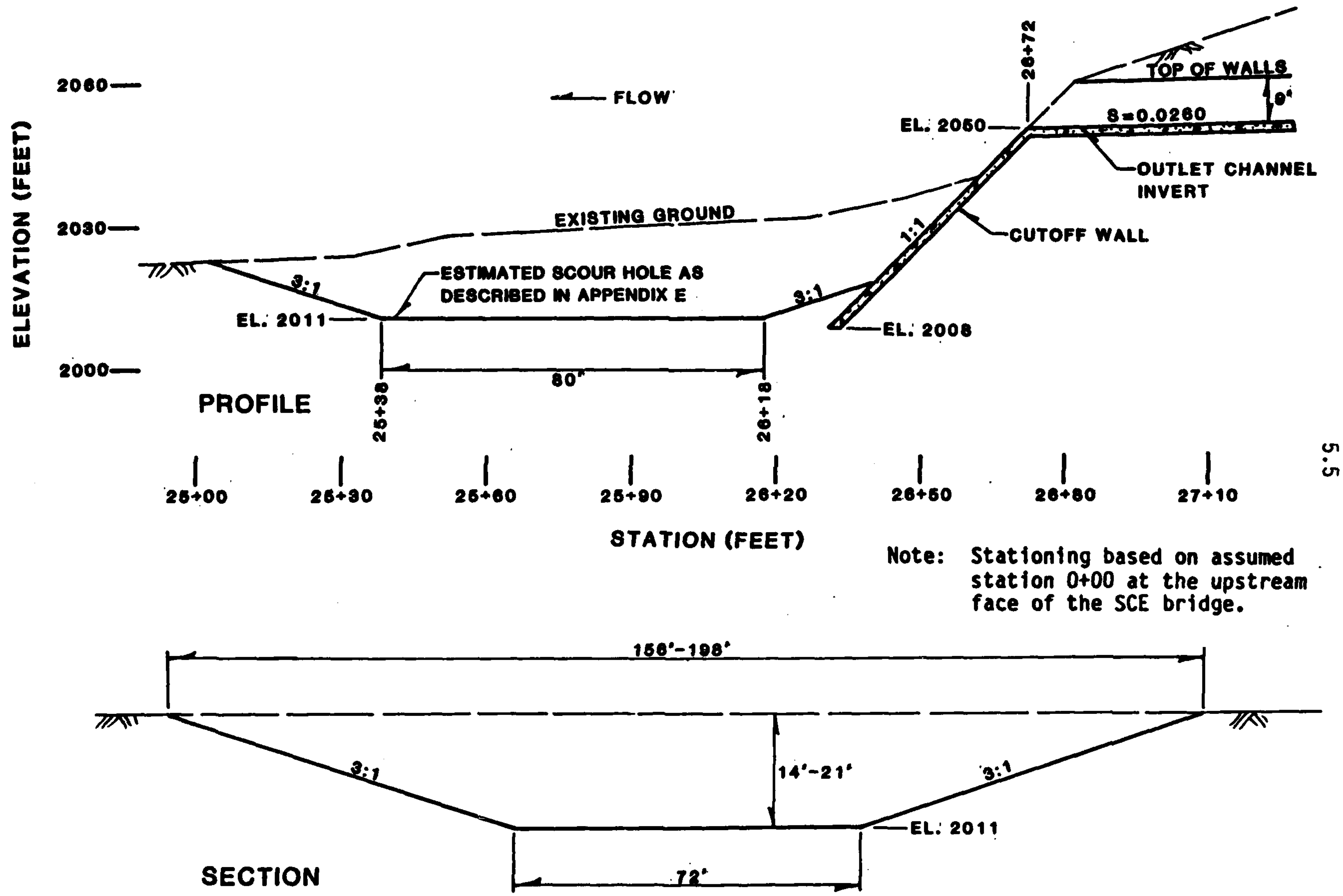


Figure 5.2. Scour hole for 2,000 cfs release.

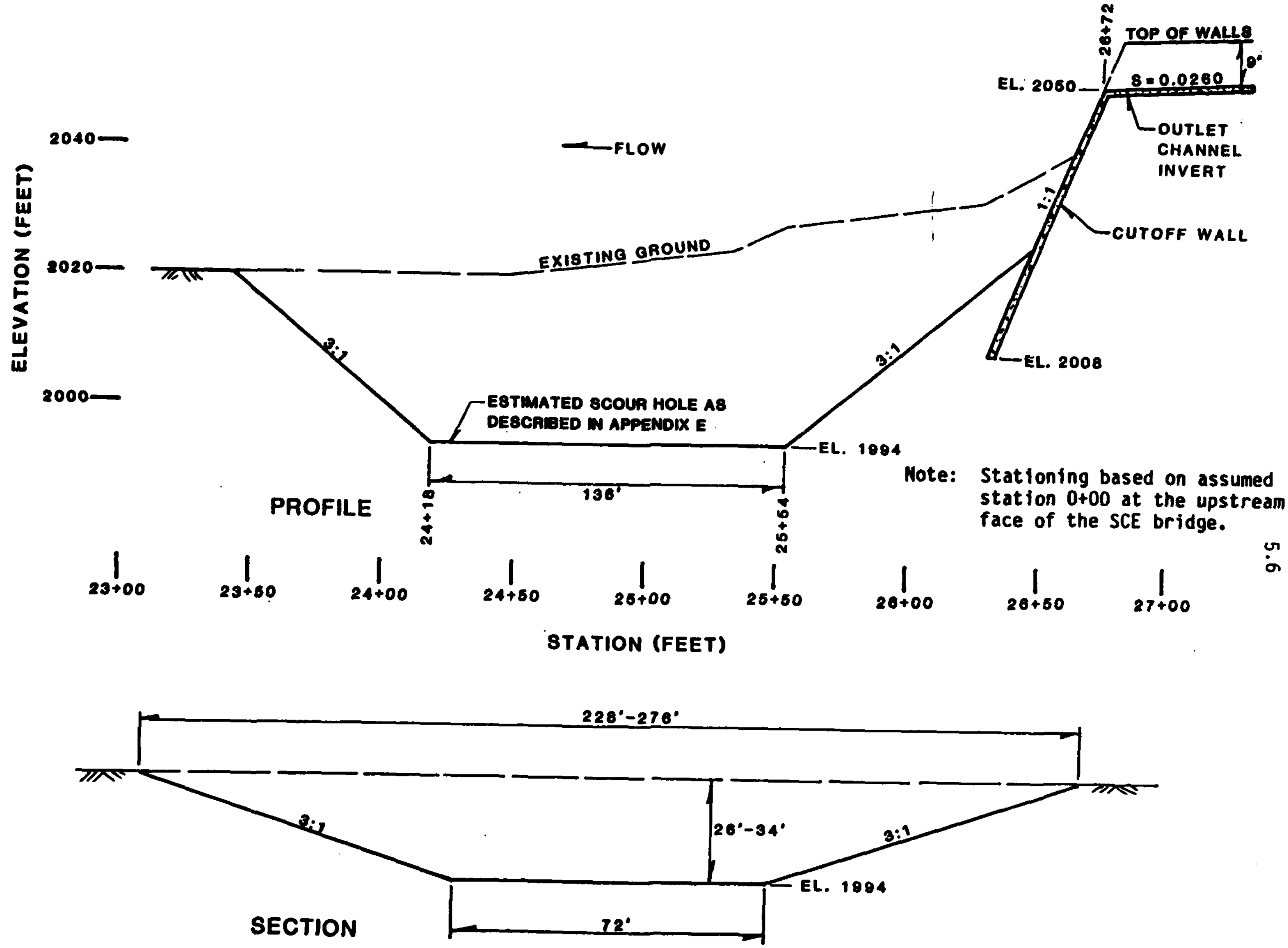


Figure 5.3. Scour hole for 8 000 cfs release

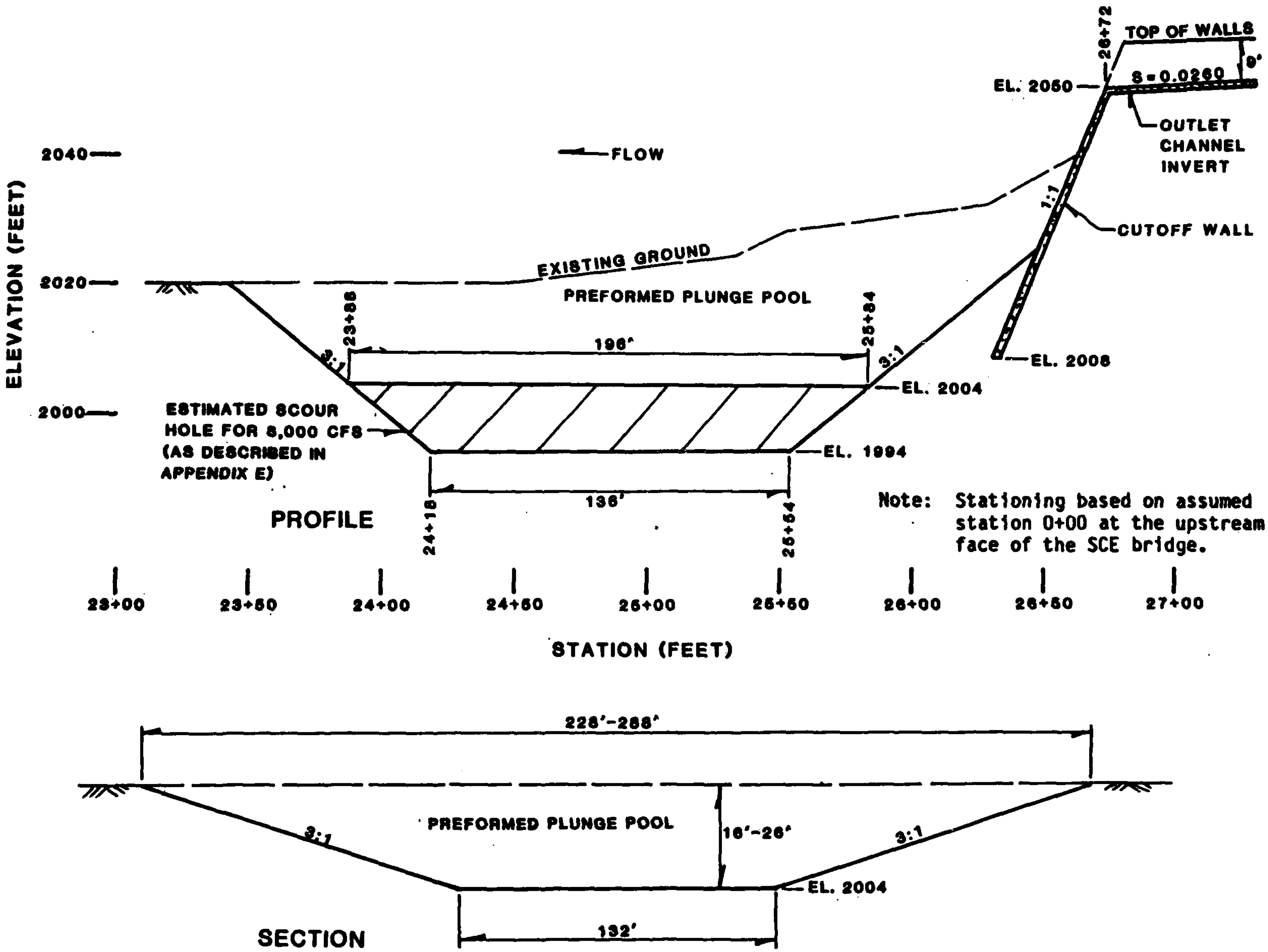


Figure 5.4. Preformed plunge pool configuration.

surface material (diameter $\geq$ 6 inches) be retained and used to line the plunge pool after excavation is complete. This will provide additional stability for the plunge pool and may reduce the ultimate scour depth. It is further suggested that the coarsest surface material (diameter >12 - 18 inches) be used to line the upstream face of the preformed plunge pool. This will provide additional protection against local scour in the vicinity of the outlet portal exit channel.

5.1.2 Toe Protection for the Outlet Portal Exit Channel

Toe protection for the outlet portal exit channel will be provided by a concrete apron and cutoff wall as indicated in Figure 4.9 and on Sheet 8. Design of the apron and cutoff wall must be adequate for all discharges up to 8,000 cfs. The scour hole profiles in Figure 4.9 indicate that 2,000 cfs is the critical discharge for the cutoff wall design. The following criteria were used to design the apron and cutoff wall:

1. The cutoff wall must extend 10 feet below the limit of the 2,000 cfs scour hole,

2. The cutoff wall must extend laterally 10 feet beyond the limits of the 2,000 cfs scour hole; i.e., 10 feet wider than the scour hole on each side,

3. The apron must transition from the outlet portal exit channel width of 18 feet at the channel to the width of the cutoff wall at existing ground, and

4. Tie-back walls, 15 feet long, must be added to both ends of the cutoff wall.

The resulting apron and cutoff wall are shown in Figure 5.5. Once the scour hole is fully developed, all but the lower 10 feet of the cutoff wall will be exposed. Therefore, the apron and cutoff wall must be anchored to prohibit failure by sliding. Because the detailed structural design of the wall is beyond the scope of this study, the cost of anchoring has not been included in the cost estimate previously given in Table 4.1. However, anchoring costs have been partially offset by over-designing the apron and cutoff wall thickness (see Figure 5.5).

Due to uncertainties involved in estimating scour depths and scour hole configurations, the cutoff wall design should not be finalized until a model study of this alternative has been conducted. In the event the model study results indicate scour depths are greater than have been estimated, the cutoff

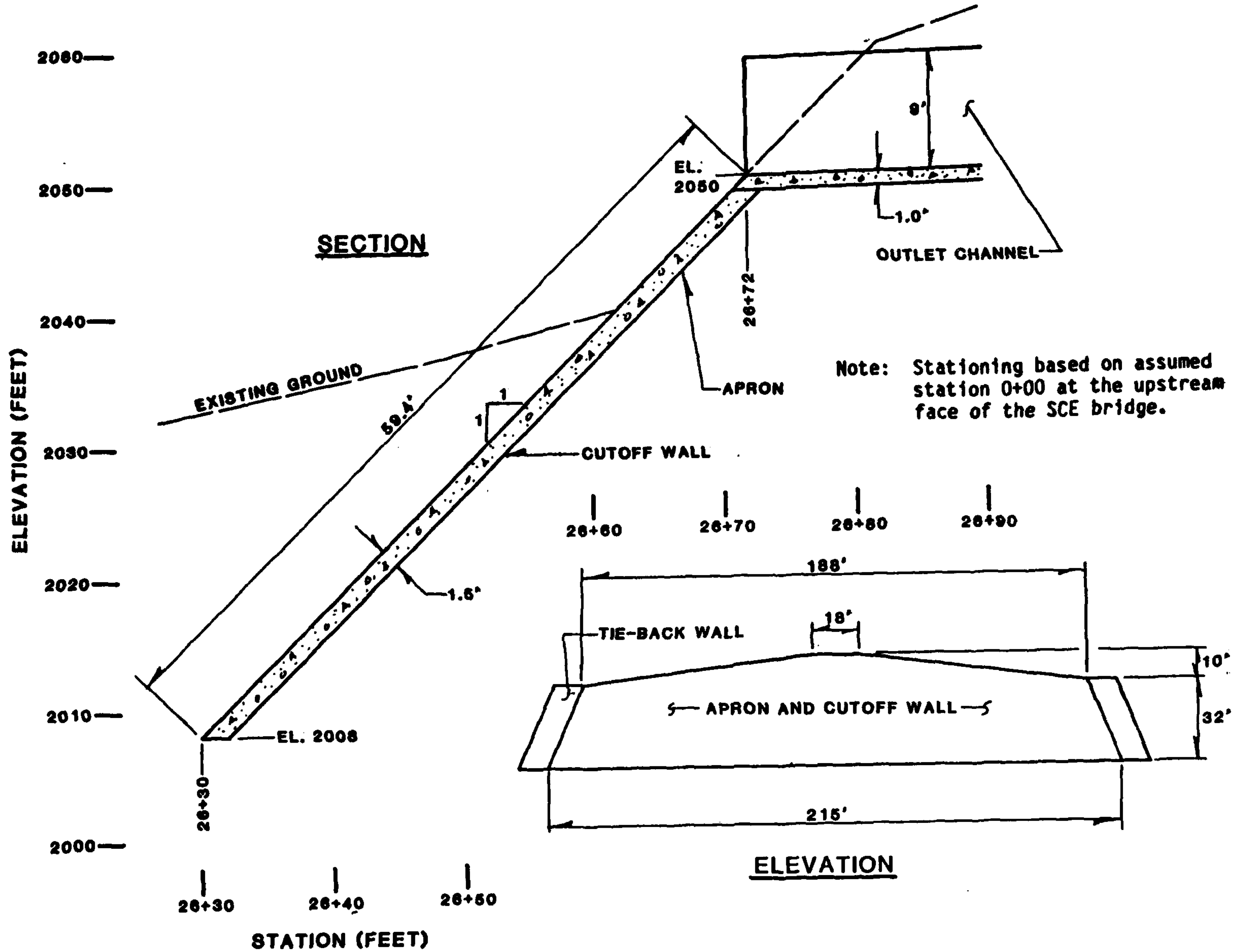


Figure 5.5. Section and elevation views of concrete apron and cutoff wall.

wall would need to be longer and extended deeper than shown in Figure 5.5. To facilitate analyzing the effect of enlarging the cutoff wall, a sensitivity analysis of alternative cost with cutoff wall depth was performed. The cost of this alternative was computed for several cutoff wall depths ranging from that required for the best-estimate scour hole to the depth required for the maximum scour hole. It was assumed that the preformed plunge pool cost remained constant for all cases. The resulting cost versus depth curve is given in Figure 5.6. The costs range from $315,000 to $913,000 for wall depths varying from approximately 42 to 86 feet. The cost-depth function is nearly linear over the range considered. Costs appear to be quite sensitive to the cutoff wall depth.

The cost of constructing the outlet portal exit channel was estimated to be $160,000 including a 20 percent contingency. Adding the cost of the outlet channel brings the total cost of the recommended design alternative for the best-estimate scour hole $475,000 (excluding the cost of anchoring the apron and cutoff wall).

5.1.3 Factor of Safety for Cutoff Wall

Although the cutoff wall shown in Figure 5.5 was designed to protect the outlet tunnel for the critical scour hole associated with only the best-estimate scour depths, a factor of safety is inherent in the design. This built-in safety factor arises from several conservative assumptions made in the scour depth and scour hole configuration calculations. These assumptions were:

1. Tailwater effects are negligible,
2. Mean diameter of the subsurface material is 15 mm (actual d_{50} of the surface-bed material is 90 mm), and
3. Scour hole sidewalls slope at 3H:1V.

To quantify the cutoff wall's factor of safety, less conservative assumptions, still within the reasonable range, were made and the scour holes reanalyzed. The new assumptions are:

1. Tailwater effects are not negligible,
2. Mean diameter of the subsurface material is 50 mm (actual d_{50} of the bed material is 90 mm), and

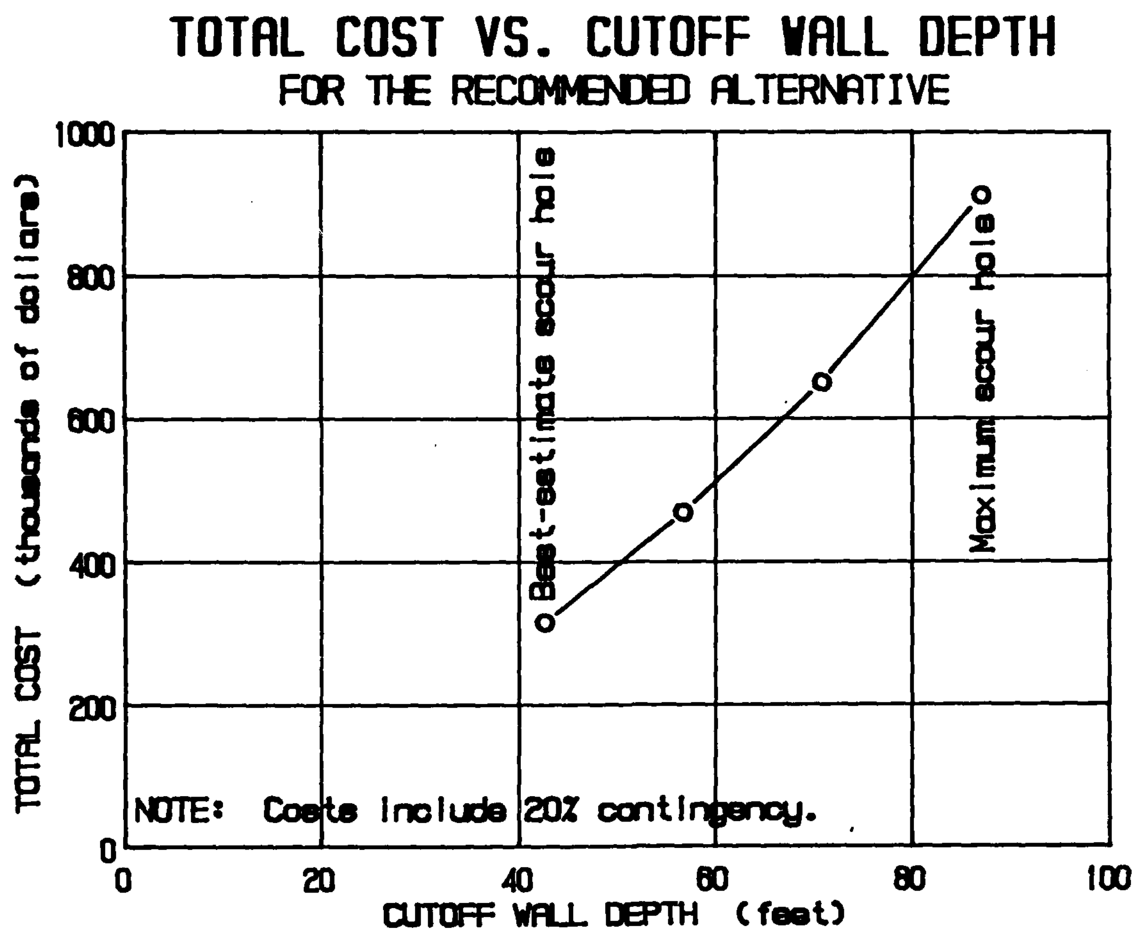


Figure 5.6. Total cost/cutoff wall depth curve for the recommended design alternative. (Total costs do not include the cost of anchoring the apron and cutoff wall.)

3. Scour hole sidewalls slope at 2H:1V (closer to the natural angle of repose).

With these assumptions, the critical scour hole is formed by the 500 cfs release. If it is assumed the cutoff wall must extend just to the limit of the 500 cfs scour hole, the elevation of the bottom of the cutoff wall must be 2,027 feet. Therefore, the required vertical depth below existing ground is approximately 13 feet. The elevation of the bottom of the design cutoff wall is 2,008 feet. Therefore, the actual vertical depth below existing ground is about 32 feet. If the factor of safety is defined as the actual cutoff wall depth divided by the required cutoff wall depth, the factor of safety is greater than 2.4. Therefore, even the cutoff wall designed based on best-estimate scour depths includes a significant factor of safety.

5.1.4 Gaging Station Location

A stream gaging station will be required to gage the reservoir releases. The gage should be located downstream of the predicted scour hole, but upstream of any significant tributaries. It is recommended that a gaging station be constructed along the east side of the canyon approximately 300 feet upstream of the existing supplementary USGS gage (as shown on Sheet 8).

5.2 Project Impacts

The following impacts would be felt by implementing the recommended design alternative.

1. The proposed Seven Oaks Dam embankment would be protected against damage due to scour.

2. The proposed outlet portal exit channel would be protected against damage due to scour.

3. A scour hole approximately 320 feet long and 280 feet wide would be formed by normal operation reservoir releases.

4. Adequate energy dissipation would be provided by the plunge pool such that the downstream channel bed would be relatively stable.

5. The potential for slope failure of the canyon walls would be reduced.

6. The potential for scour damage to all structures, pipelines, and other facilities in the Santa Ana Canyon would be reduced or eliminated.

7. The frequency and extent of flooding of the endangered species would be reduced, but the Eriastrum would still be subject to flooding on a regu-

lar basis. (If channelization measures are employed, flooding of the Eriastrum could be completely eliminated, if desired).

8. Postproject flooding downstream of the dam site would be reduced relative to preproject flooding because postproject releases would be substantially smaller than preproject discharges.

9. The significant armoring potential of the river channel between the dam site and Greenspot Road, and the reduction in flows downstream of the dam site would result in reduced scour potential for postproject conditions.

VI. SPILLWAY FLOW IMPACTS

If the proposed Seven Oaks Dam embankment should be threatened by failure due to overtopping, the emergency spillway would release flow to Deep Creek until the danger had passed. According to information provided by the COE, a 220-year event or greater would require emergency releases. Although these releases would occur infrequently, their magnitudes would be substantial. The proposed operating schedule gave two points on the discharge-elevation curve for pool elevations greater than the emergency spillway crest elevation. The releases associated with pool elevations 20 and 30 feet above the spillway crest are 130,000 and 240,000 cfs, respectively.

The postproject impacts on Deep Creek and the Santa Ana River were investigated by analyzing the armoring potential for the two given discharges at the confluence. A supercritical HEC-2 analysis was performed for Deep Creek from the spillway to the Santa Ana River. The flow velocities at the confluence were found to range from about 50 to 60 feet per second. The armoring calculations revealed that the critical particle diameter for 130,000 and 240,000 cfs at the confluence would be 11 and 14 feet, respectively. Therefore, the channel bed would not armor for emergency operation conditions.

Estimating the potential depth of scour would require information concerning the actual magnitude and duration of the emergency releases. Since this information is not available at this time, it can only be concluded that scour damage to both Deep Creek and the Santa Ana River would be substantial for emergency operation conditions.

It should be assumed that the three pipelines which cross Deep Creek between the spillway and the Santa Ana River (the Greenspot Pipeline, the Intermediate Line, and the Bear Valley Highline) would be severely damaged by emergency releases. However, due to the severity and infrequency of scour damage caused by emergency releases, it may be more economical to replace the damaged pipelines following emergency operations rather than attempt to provide protection for such extreme events.

Potential impacts to the dam embankment due to emergency spillway releases are thought to be minimal due to the distance between the Deep Creek confluence and the dam (approximately 4,000 feet) and the significant armoring potential of the Santa Ana River channel in that reach.

VII. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are provided as a brief summary of results for the channel stabilization design and river sediment transport study for Seven Oaks Dam.

1. Downstream of Greenspot Road, the Seven Oaks Dam impact on aggradation/degradation trends is minor. For preproject conditions, the maximum deposition or scour depth for the SPF would be less than 1.5 feet. While the maximum SPF deposition or scour depth would be slightly larger for postproject conditions, it would still be less than 1.5 feet. Further, the maximum average annual scour depths for pre- and postproject conditions would be less than 0.2 and 0.4 feet, respectively.

2. Upstream of Greenspot Road, the channel is essentially armored for postproject flows. Therefore, even though the upstream sediment supply would be eliminated, the maximum scour depth in this reach would be less than 1.5 feet.

3. The recommended design alternative for energy dissipation of normal operation releases is Alternative 2, preformed plunge pool and toe protection for the outlet portal exit channel. The toe protection would be a concrete apron and a sloping cutoff wall located at the downstream end of the outlet portal exit channel.

4. The outlet portal exit channel would be a 194-foot-long, 18-foot-wide rectangular reinforced concrete channel extending from the outlet tunnel portal to the daylight line at elevation 2,050 feet.

5. The cost of constructing the outlet portal exit channel, preforming the plunge pool, and constructing the concrete apron and cutoff wall (neglecting anchoring) was estimated to be $475,000.

6. Although several conservative assumptions were made when estimating scour depths and scour hole configurations, accurately predicting scour hole geometry is difficult. Therefore, it is strongly recommended that a carefully conducted model study be performed before finalizing this design.

7. The endangered species in the Santa Ana Canyon is currently being partially flooded on a frequent basis. For postproject conditions, with the recommended alternative in place, the extent and frequency of flooding of the endangered species would be reduced. However, flooding would still occur on a fairly regular basis.

VIII. REFERENCES AND DATA LIST

8.1 References

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24. Water Resources Data, California, Water Year 1983, U.S. Geological Survey, Vol. 1, 1985.

25. Water Resources Data, California, Water Year 1984, U.S. Geological Survey, Vol. 1, 1986.

8.2 Data List

1. U.S. Geological Survey, Quadrangle Maps, Dated 1967 and Photorevised 1980 (except where noted):

   Prado Dam, California (1967, Photorevised 1981)
   Corona North, California (1967, Photorevised 1981)
   Riverside West, California
   Riverside East, California
   Fontana, California
   San Bernardino South, California
   San Bernardino North, California
   Redlands, California
   Harrison Mountain, California
   Yucaipa, California
   Keller Peak, California (1967, Photoinspected 1978)
   Big Bear Lake, California (1970)

2. Topographic Mapping (1:2400 scale), San Bernardino County, Department of Transportation and Flood Control, Dated December 1964:

   T.1S, R.2W, Sections 4, 6, 7, 8, 9, 17, 18
   T.2S, R.5W, Sections 1, 2

3. Topographic Mapping (1:2400 scale), T.2S, R.5W, Section 11, Riverside County Flood Control and Water Conservation District, Dated February 1965.

4. Topographic Mapping (1:2400 scale), Santa Ana River Upstream of Greenspot Road, U.S. Army Corps of Engineers, Dated 1984.

5. Santa Ana River Profile, Prado Dam to Seven Oaks Dam Site, U.S. Army Corps of Engineers, taken from USGS Quadrangle Maps, September 1986.

6. Plan and Profile North Section of Intermediate Line (DWG. No. 4708), dated September 1947, Bear Valley Mutual Water Company, provided by San Bernardino Valley Municipal Water District.

7. Greenspot Pipeline, Phase II Plan & Profile (DWG. Nos. 45003, 45004, & 45005), dated December 1984, San Bernardino Valley Municipal Water District.

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9. Plans for the Santa Ana Weir and SBCWCD Canal and Headworks, Dated May 1930, San Bernardino County Water Conservation District.

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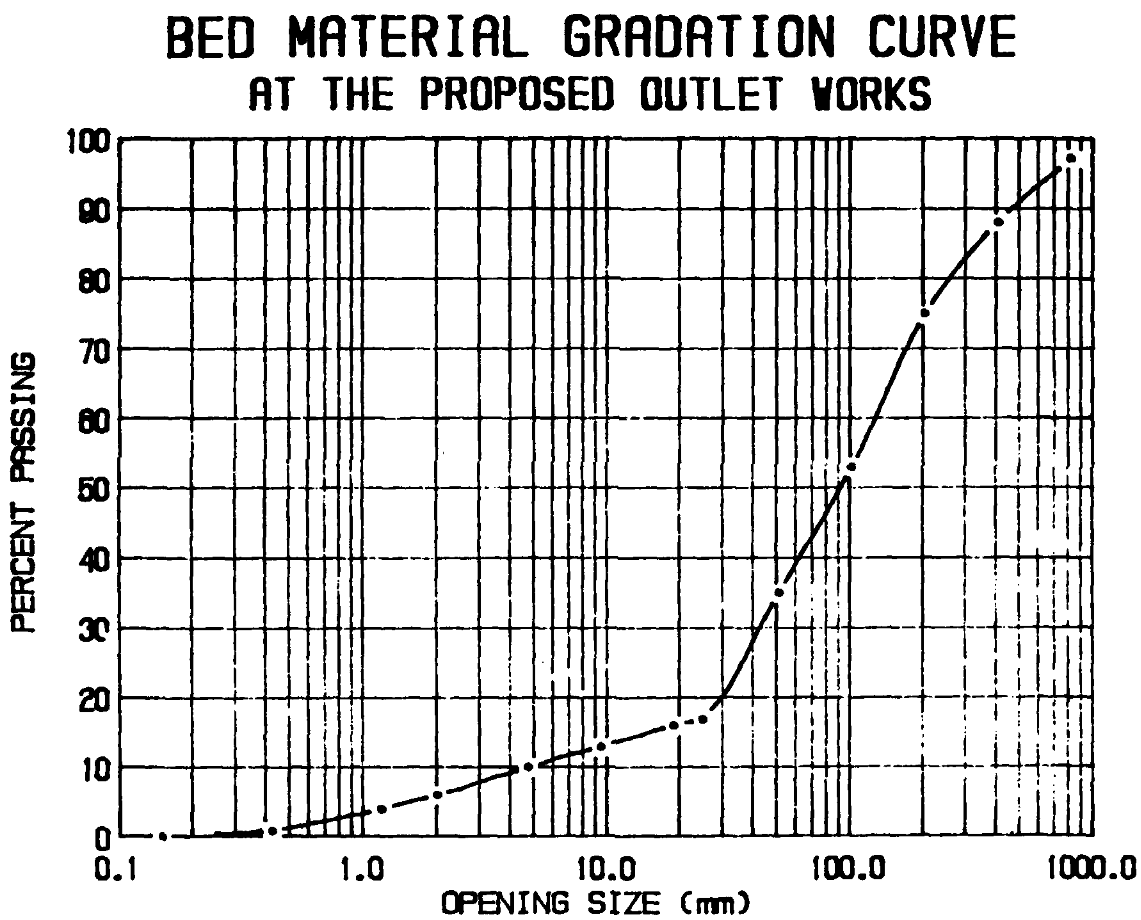
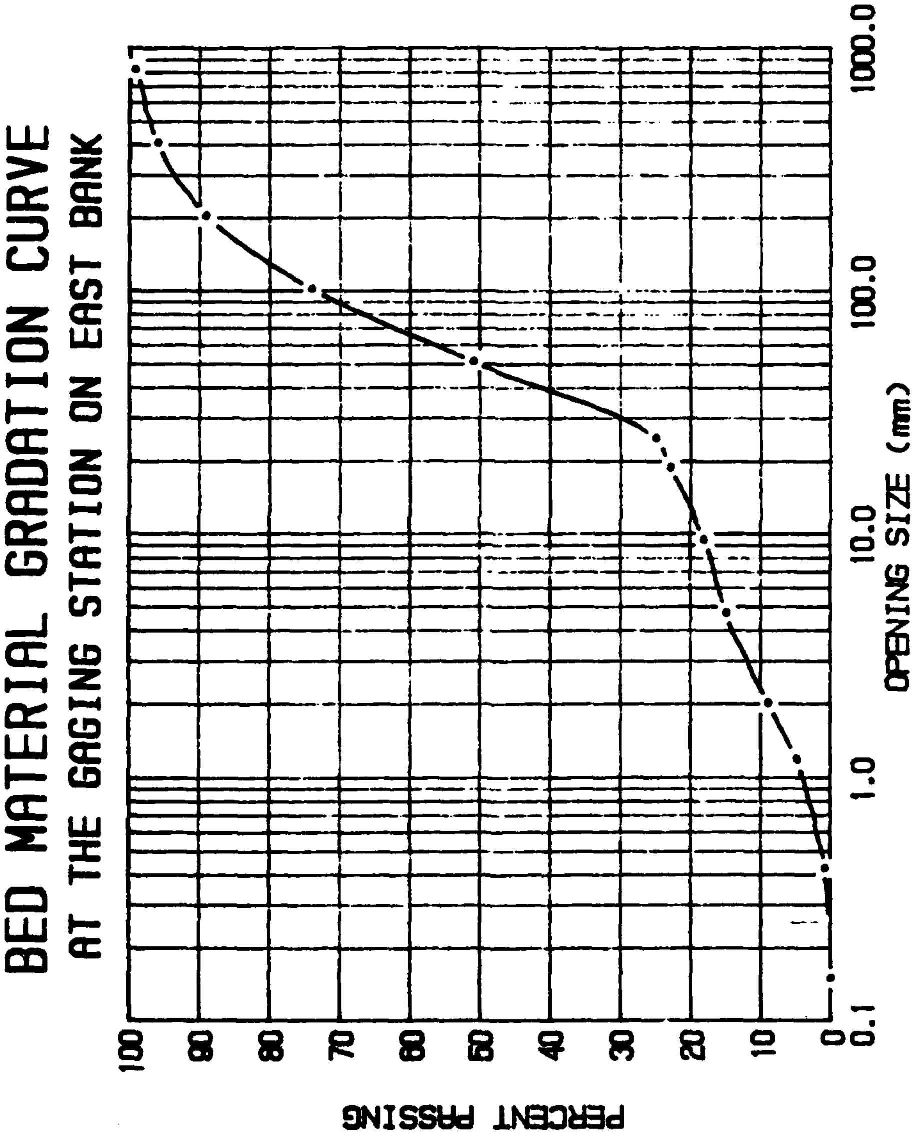
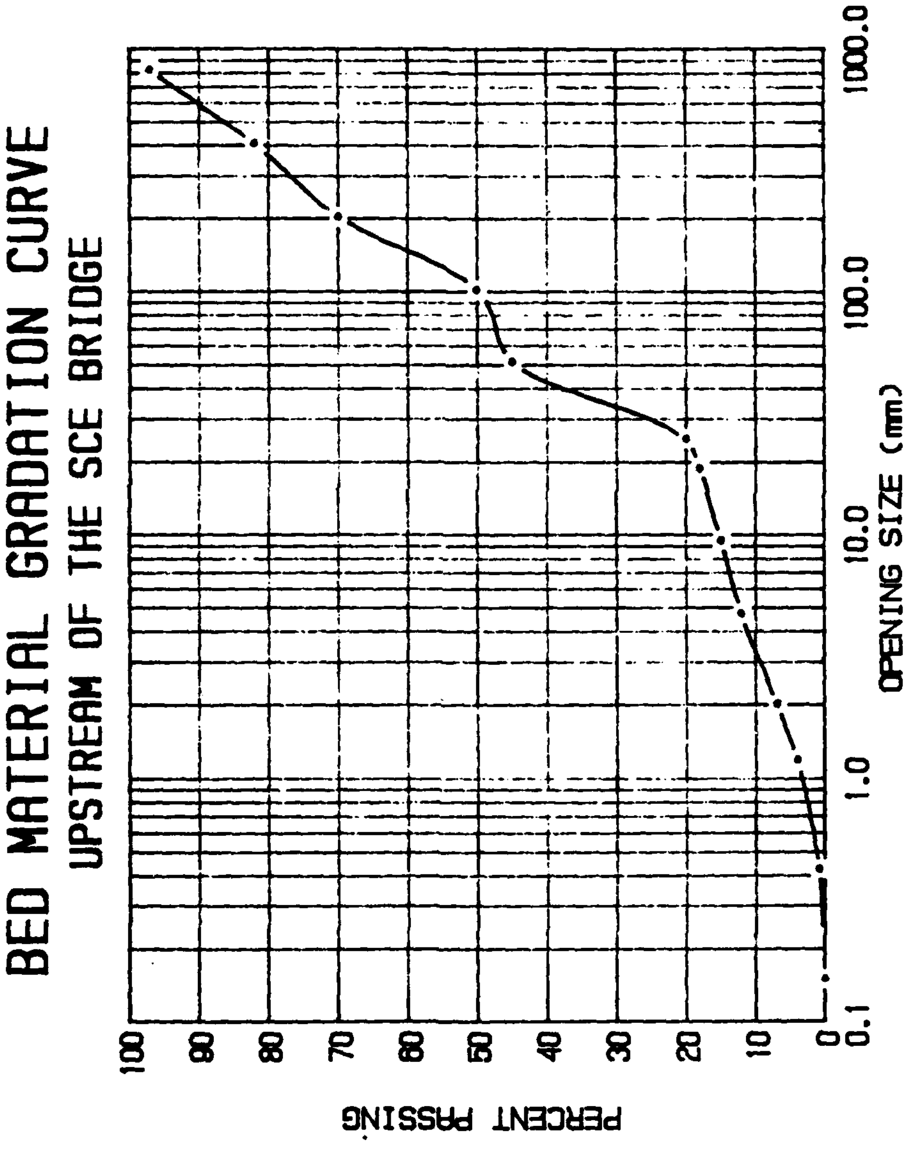
APPENDIX A

BED-MATERIAL GRADATION CURVES

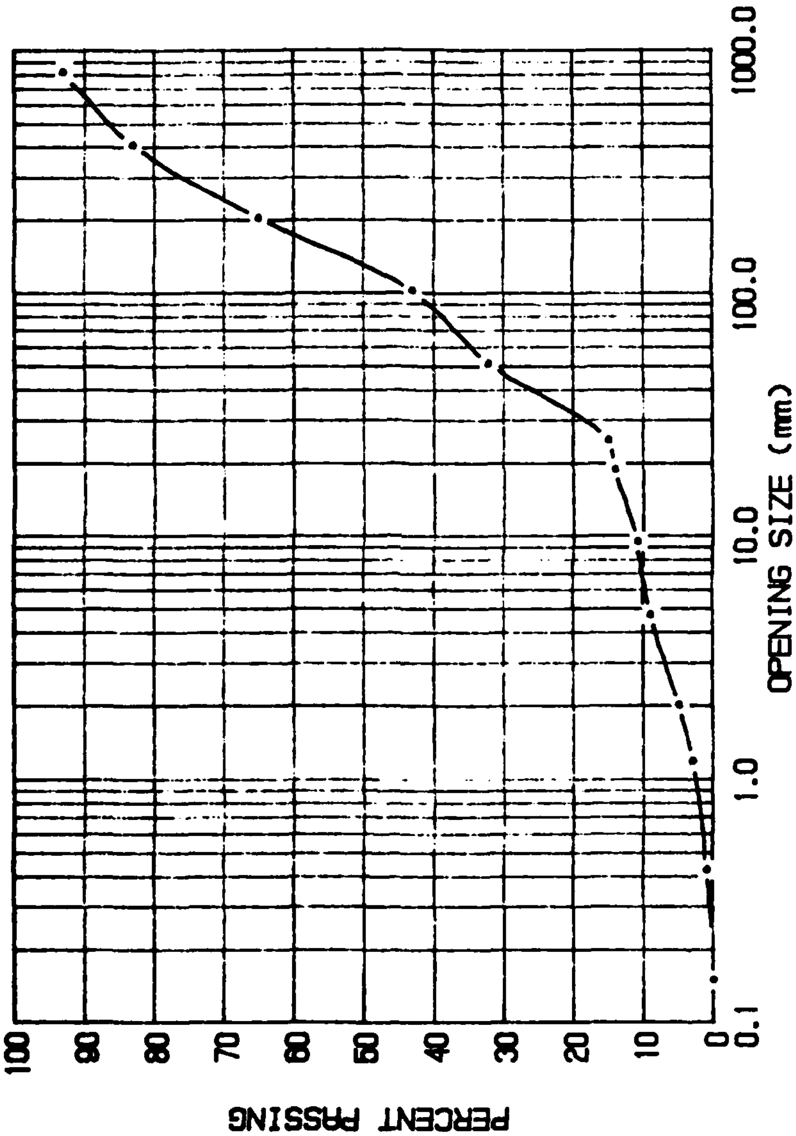

BED MATERIAL GRADATION CURVE
AT THE PROPOSED OUTLET WORKS
PERCENT PASSING
0
10
20
30
40
50
60
70
80
90
100
0.1
1.0
10.0
100.0
1000.0
OPENING SIZE (mm)

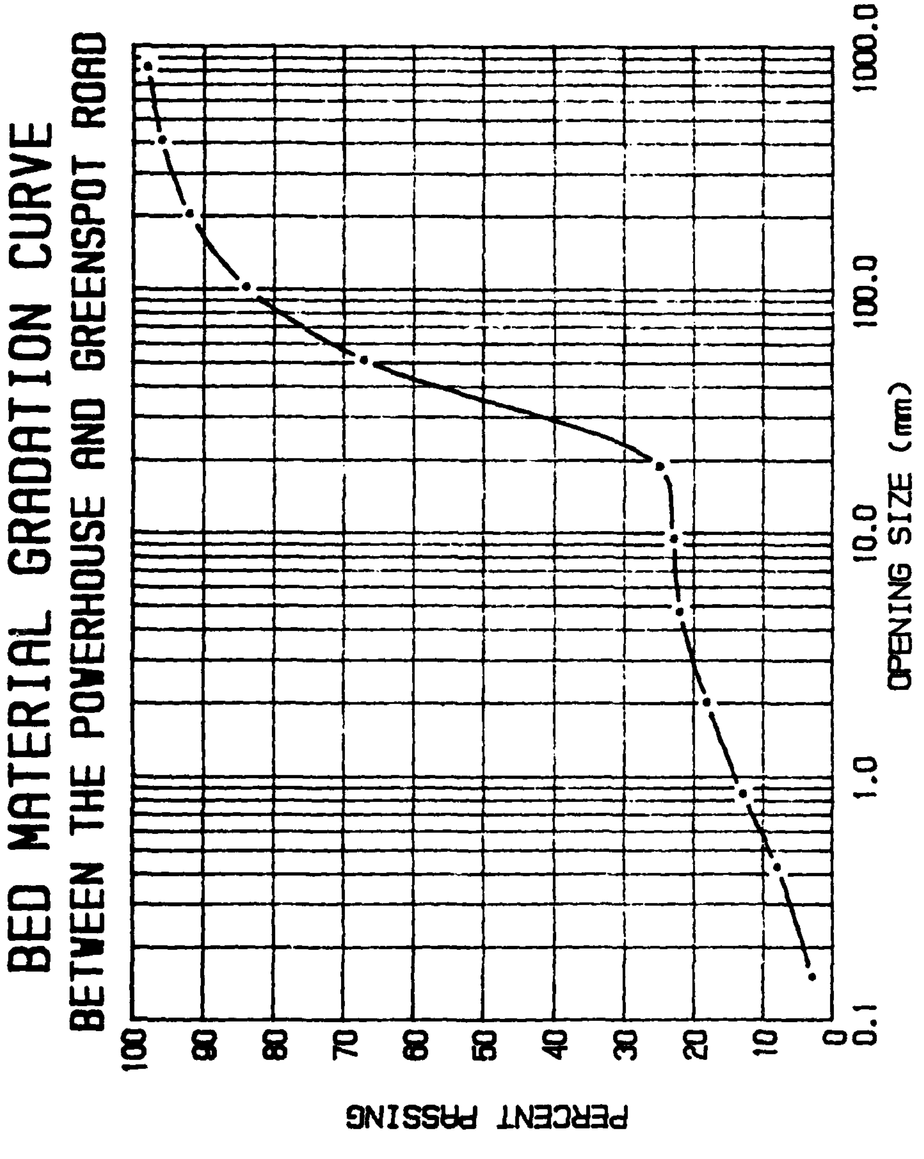

BED MATERIAL GRADATION CURVE
AT THE GAGING STATION ON EAST BANK
PERCENT PASSING
100
90
80
70
60
50
40
30
20
10
0
0.1
1.0
10.0
100.0
1000.0
OPENING SIZE (mm)


BED MATERIAL GRADATION CURVE
UPSTREAM OF THE SCE BRIDGE
PERCENT PASSING
100
90
80
70
60
50
40
30
20
10
0
OPENING SIZE (mm)
0.1
1.0
10.0
100.0
1000.0

BED MATERIAL GRADATION CURVE

DOWNSTREAM OF THE SCE FLUME




BED MATERIAL GRADATION CURVE
BETWEEN THE POWERHOUSE AND GREENSPOT ROAD
PERCENT PASSING
100
90
80
70
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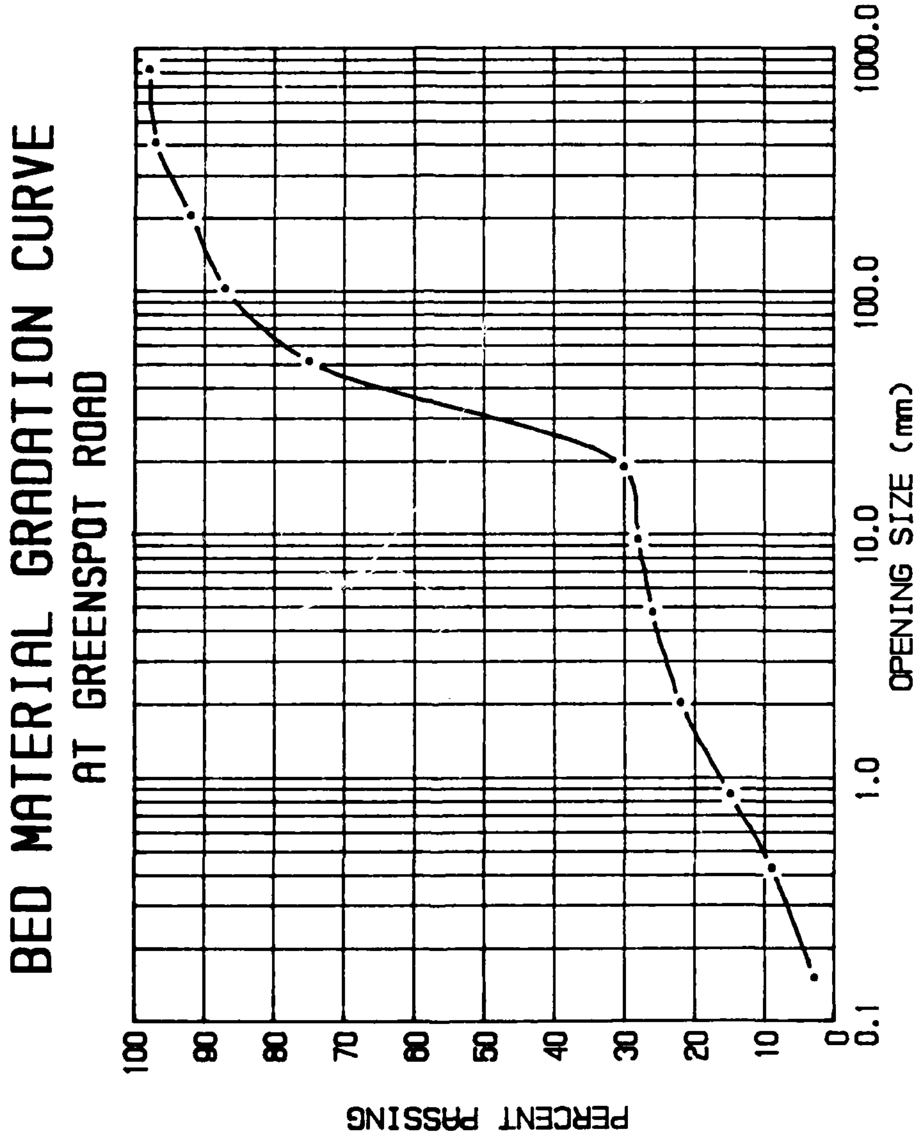
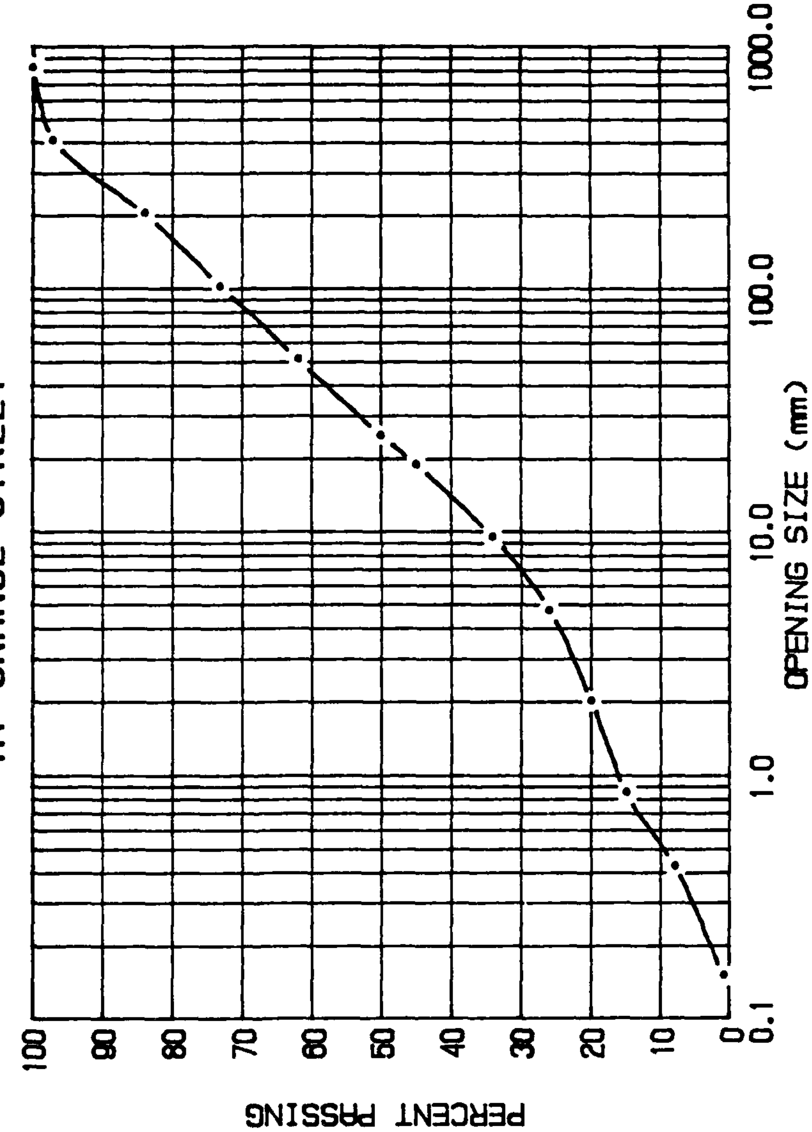
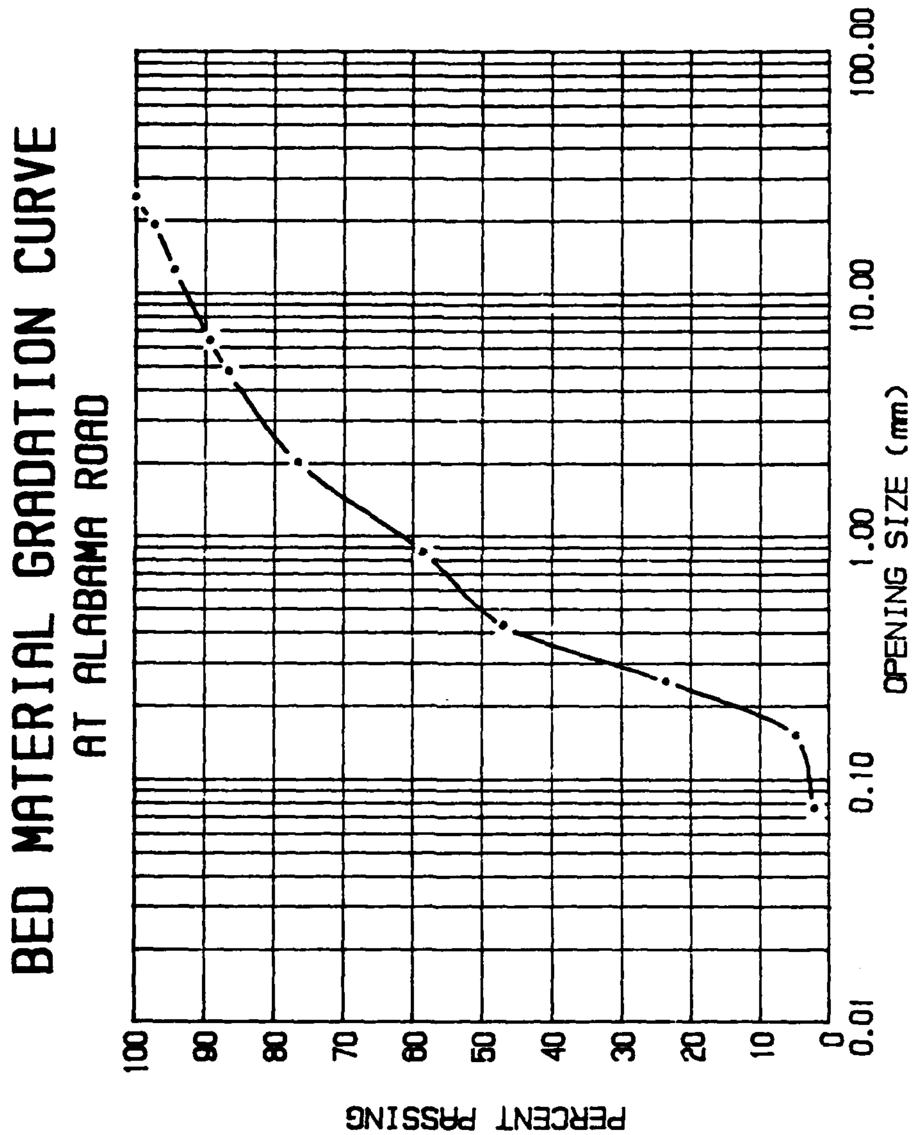
10.0
100.0
1000.0
OPENING SIZE (mm)


BED MATERIAL GRADATION CURVE
AT GREENSPOT ROAD
PERCENT PASSING
100
90
80
70
60
50
40
30
20
10
0
0.1
1.0
10.0
100.0
1000.0
OPENING SIZE (mm)

BED MATERIAL GRADATION CURVE
AT ORANGE STREET




BED MATERIAL GRADATION CURVE
AT ALABAMA ROAD
PERCENT PASSING
100
90
80
70
60
50
40
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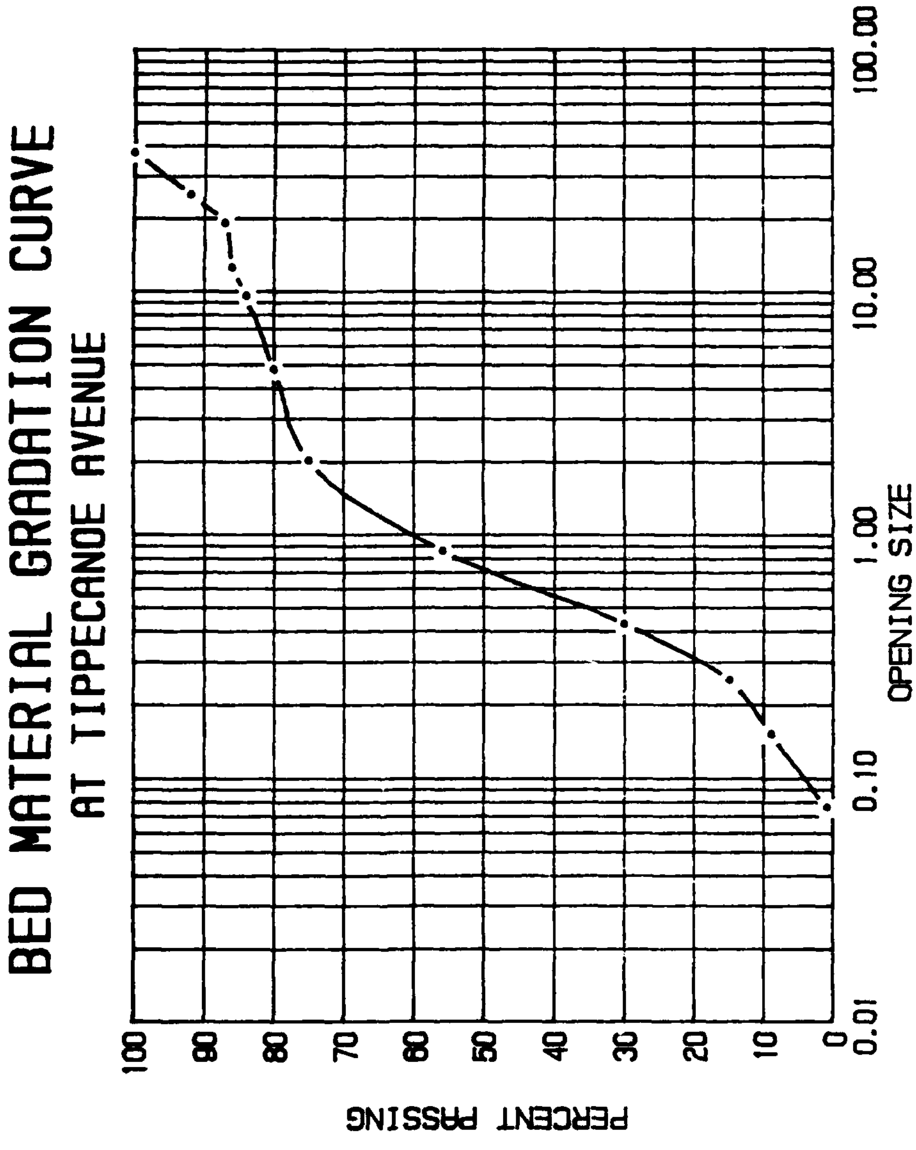
10
0
0.01
0.10
1.00
10.00
100.00
OPENING SIZE (mm)


BED MATERIAL GRADATION CURVE
AT TIPPECANOE AVENUE
PERCENT PASSING
100
90
80
70
60
50
40
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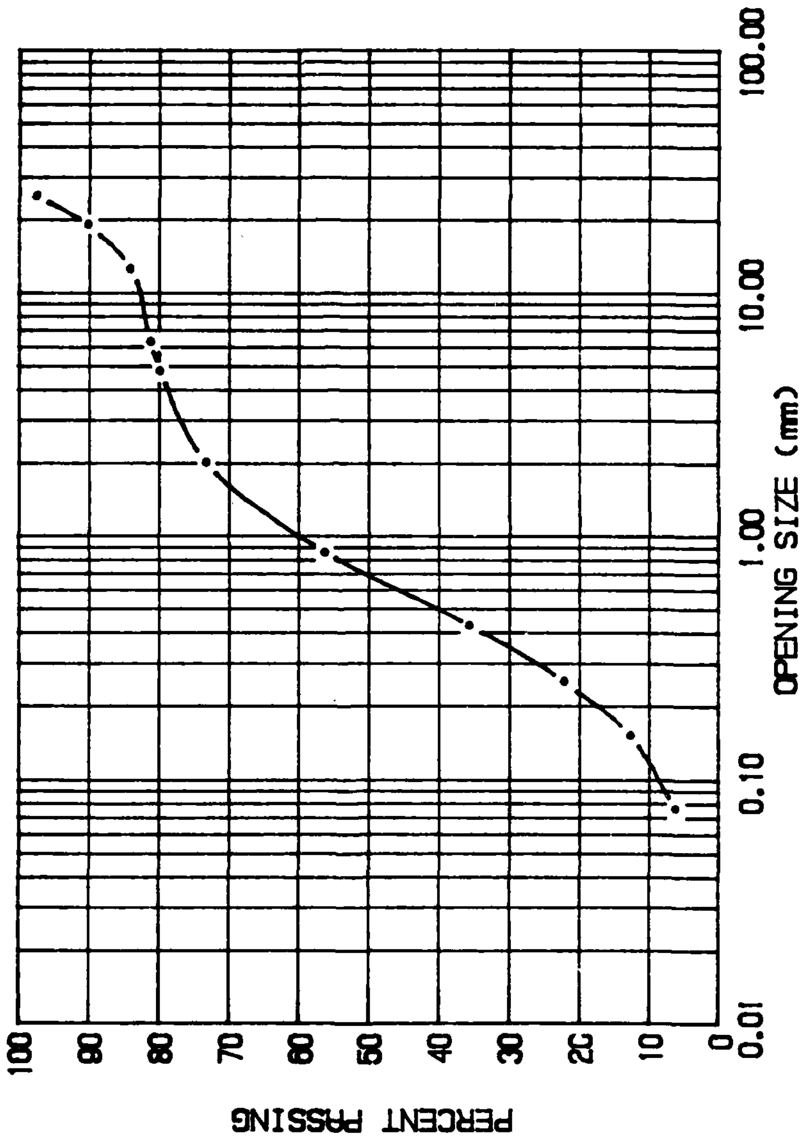
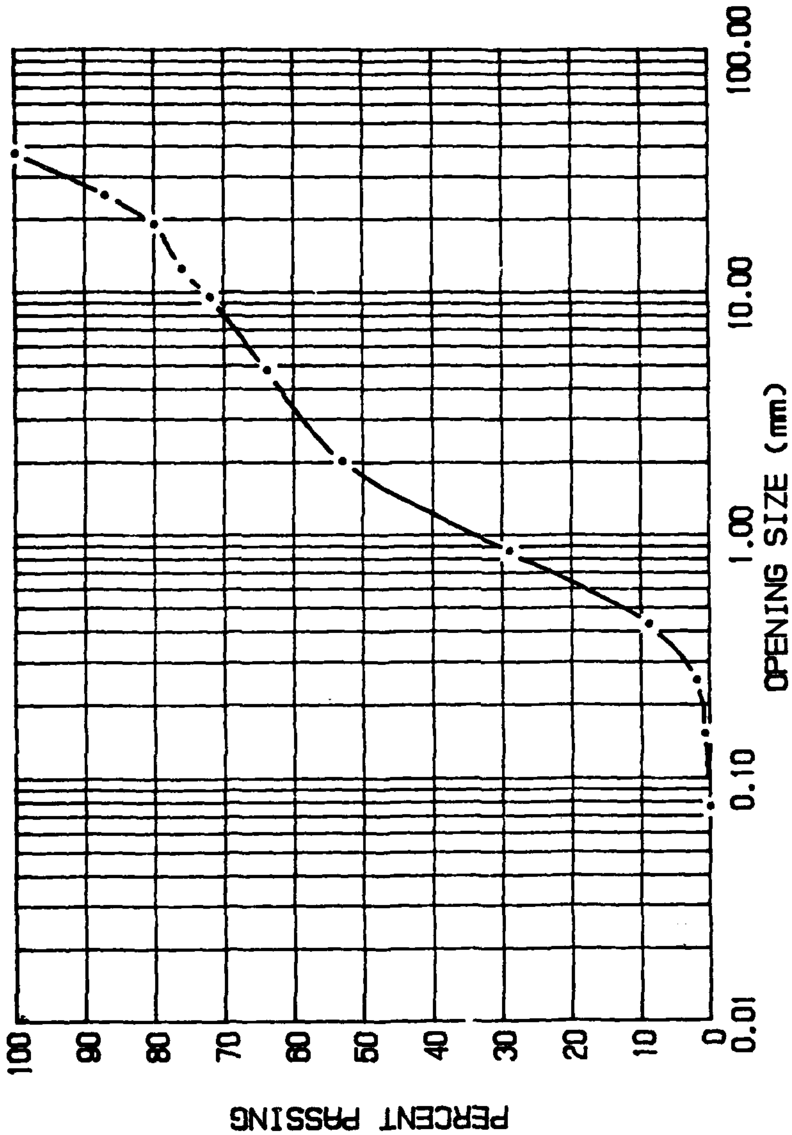
0
0.01
0.10
1.00
10.00
100.00
OPENING SIZE

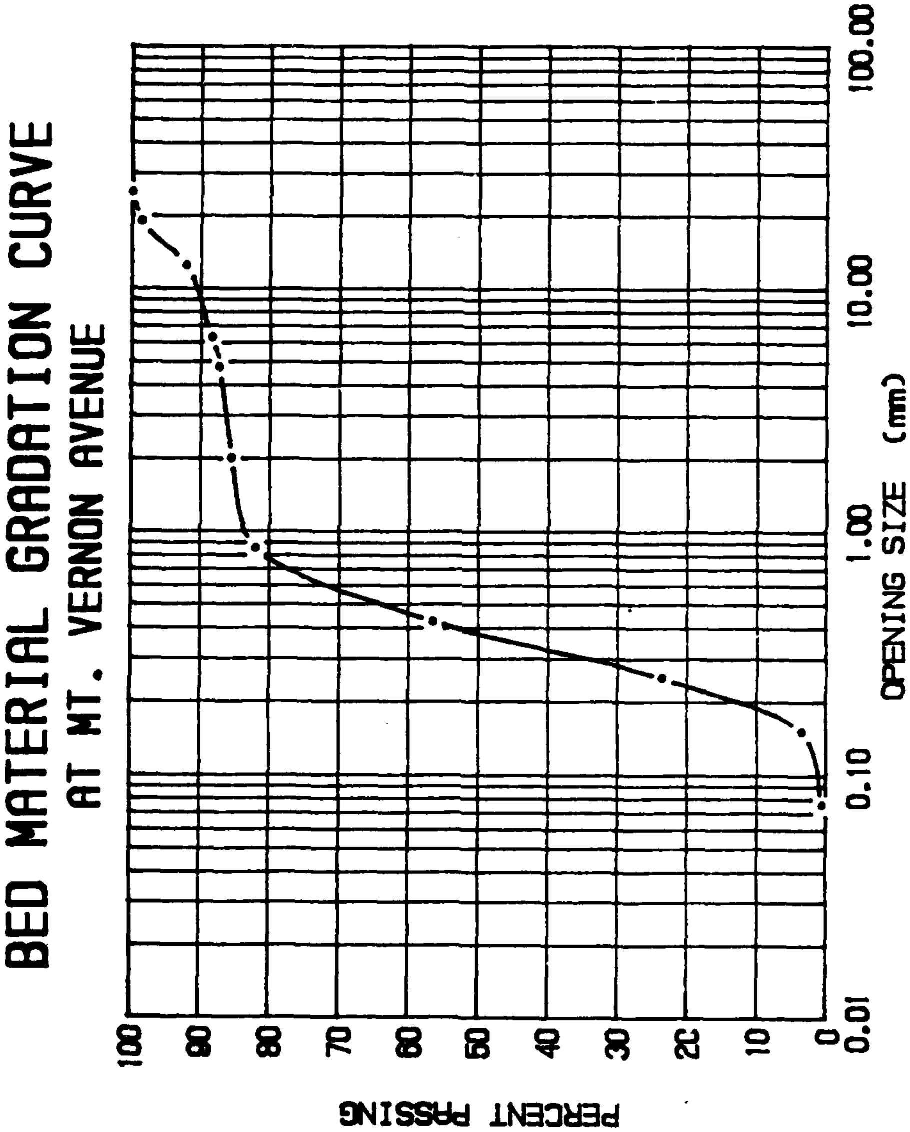
BED MATERIAL GRADATION CURVE

AT WATERMAN AVENUE



BED MATERIAL GRADATION CURVE
AT E STREET




BED MATERIAL GRADATION CURVE
AT MT. VERNON AVENUE
PERCENT PASSING
100
90
80
70
60
50
40
30
20
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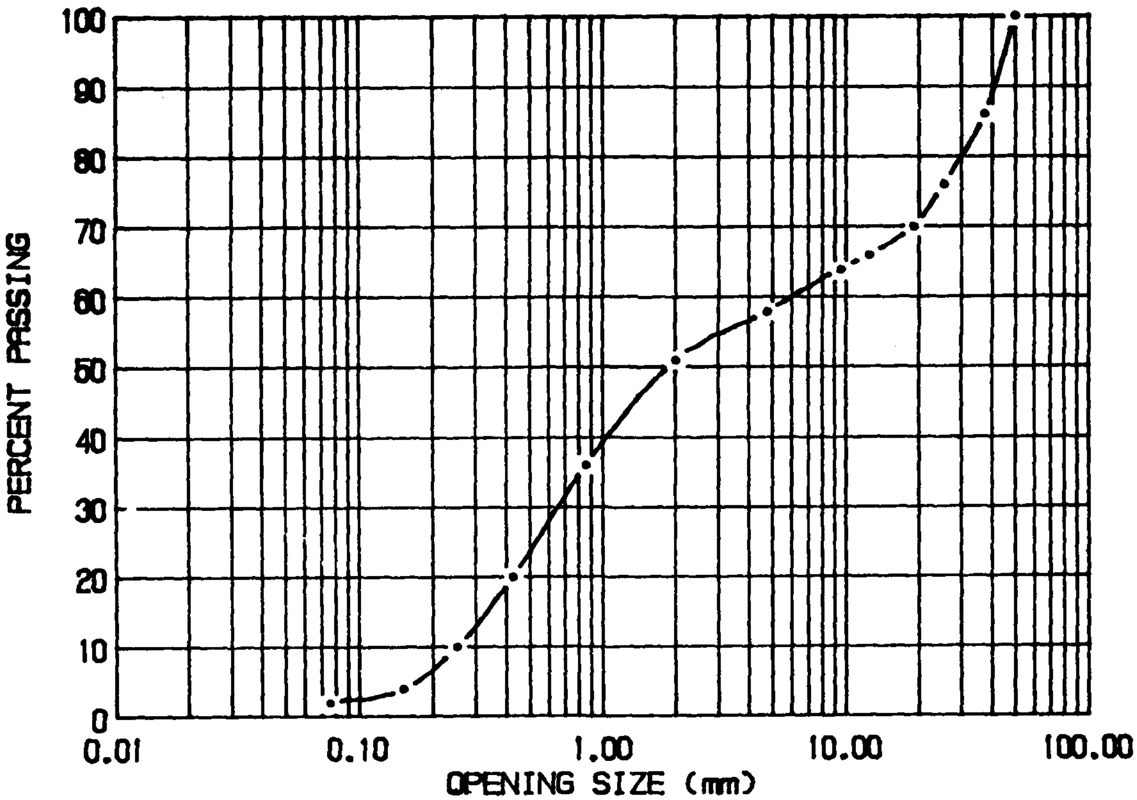
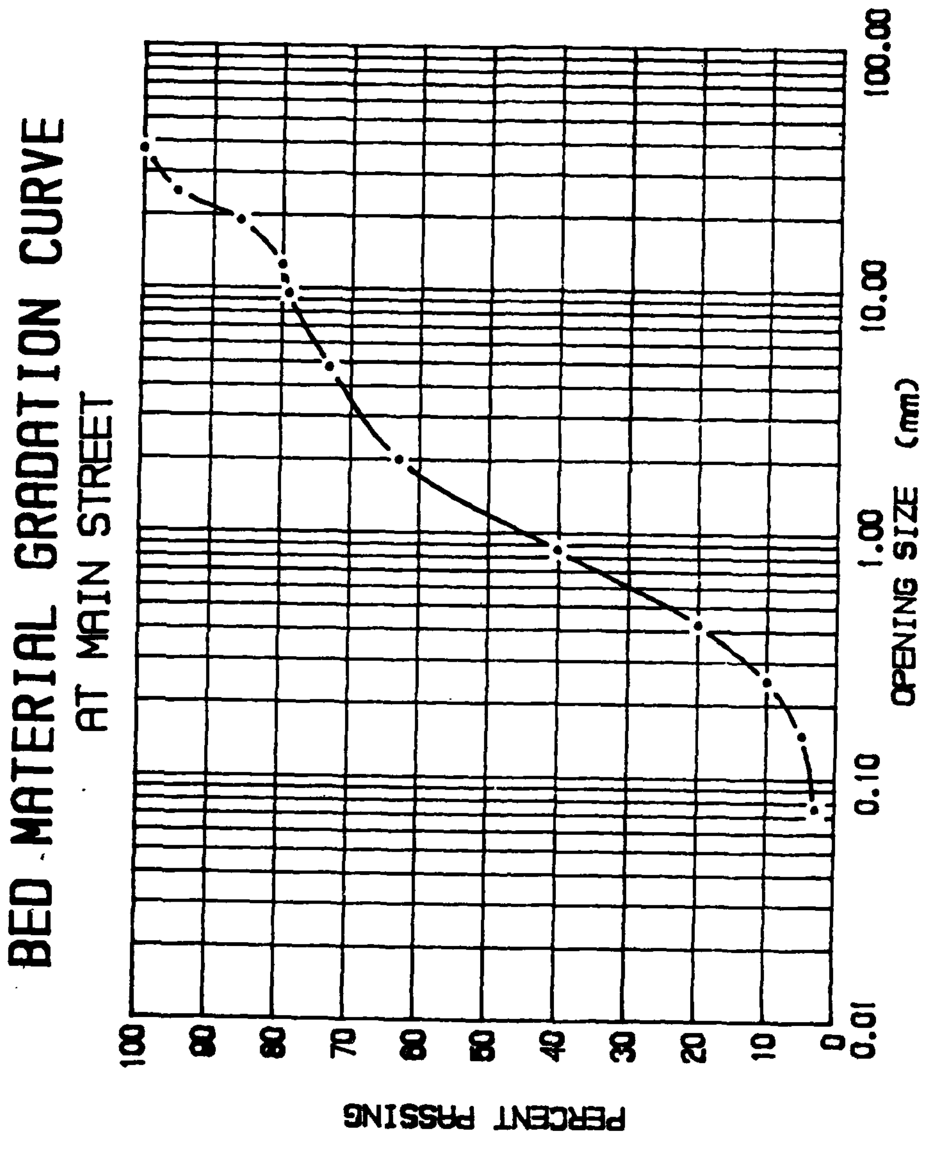
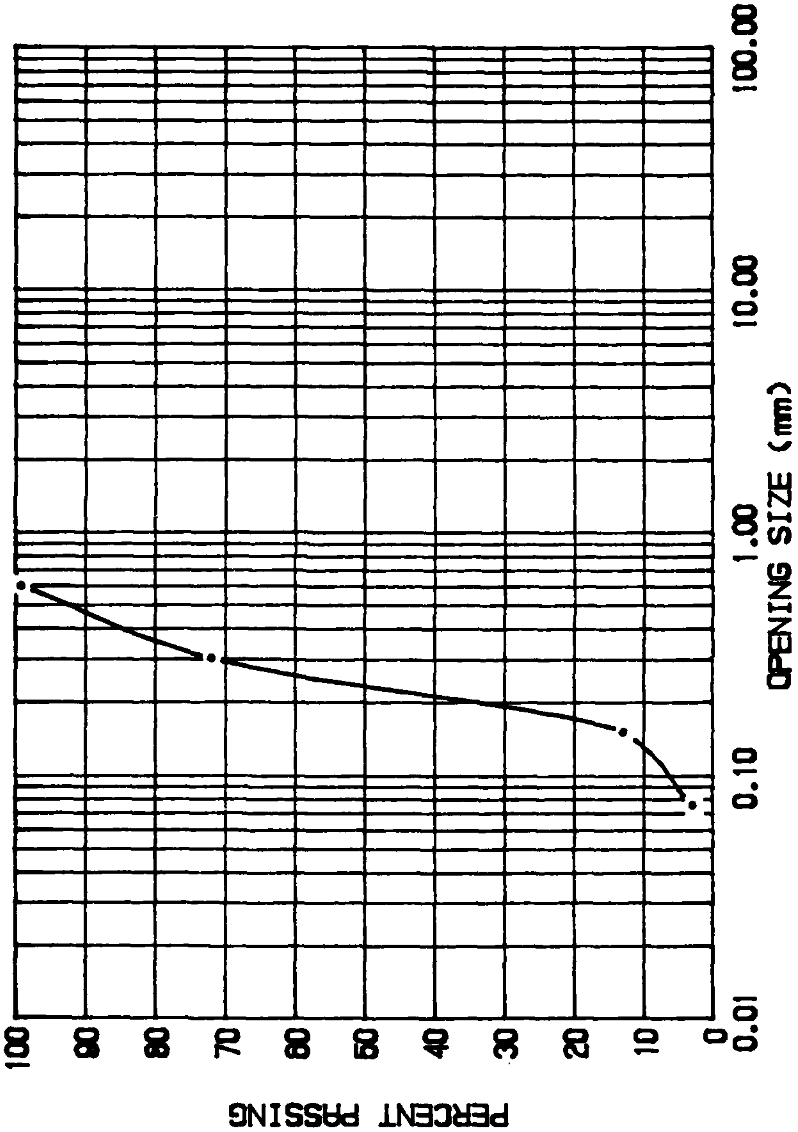
0.10
1.00
10.00
100.00
OPENING SIZE (mm)

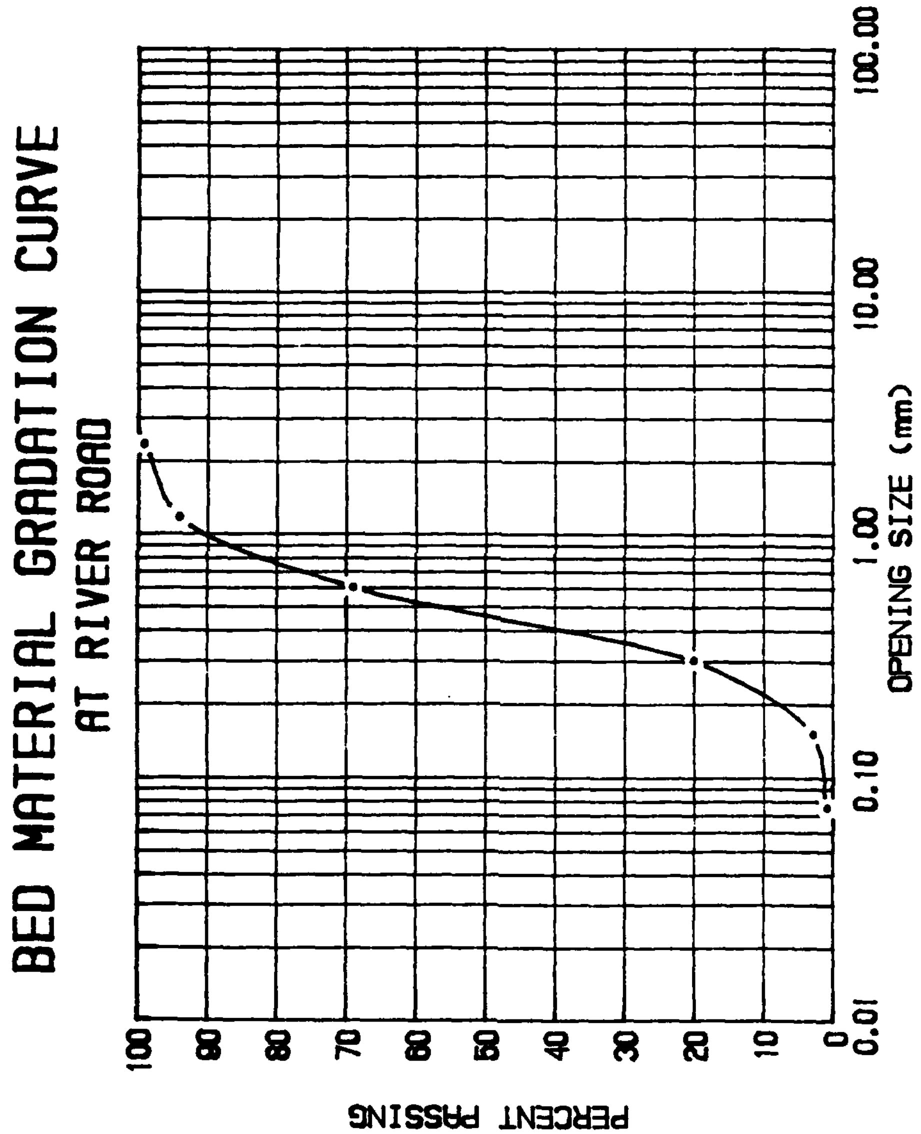
BED MATERIAL GRADATION CURVE
AT 8TH STREET




BED MATERIAL GRADATION CURVE
AT MAIN STREET
PERCENT PASSING
100
90
80
70
60
50
40
30
20
10
0
0.01
0.10
1.00
10.00
100.00
OPENING SIZE (mm)

BED MATERIAL GRADATION CURVE
AT HAMNER AVENUE




BED MATERIAL GRADATION CURVE
AT RIVER ROAD
PERCENT PASSING
100
90
80
70
60
50
40
30
20
10
0
0.01
0.10
1.00
10.00
100.00
OPENING SIZE (mm)

APPENDIX B

HYDRAULIC ANALYSIS RESULTS

Table B.1. Reach Averaged Hydraulic Parameters for the Main Channel of the Santa Ana River (Case 1, Preproject Conditions).

Reach	Length (mi)	Hydraulic Depth (ft)	Velocity (fps)	Effective Width (ft)
		STANDARD PROJECT FLOOD		
1	1.0	9.0	14.6	334
2	5.5	5.0	12.1	862
3	1.9	4.3	9.0	2,144
4	4.3	10.7	10.7	858
5	4.8	14.4	11.5	1,173
6	4.5	15.0	15.2	982
7	2.0	7.3	6.7	2,530
8	3.5	12.3	12.9	941
9	3.0	8.3	7.8	2,469
10	5.0	19.0	15.0	912
		100-YEAR FLOOD		
1	1.0	8.2	13.8	313
2	5.5	4.2	11.5	783
3	1.9	3.5	8.1	2,077
4	4.3	8.2	9.7	775
5	4.8	12.2	10.7	1,079
6	4.5	13.4	14.2	922
7	2.0	6.0	6.8	2,287
8	3.5	11.1	11.9	895
9	3.0	7.3	7.3	2,366
10	5.0	17.0	14.3	855
		50-YEAR FLOOD		
1	1.0	6.3	11.7	267
2	5.5	3.1	10.1	667
3	1.9	2.6	6.8	1,971
4	4.3	6.0	8.4	681
5	4.8	8.7	9.3	975
6	4.5	9.9	12.2	851
7	2.0	4.5	5.9	1,905
8	3.5	8.8	10.0	795
9	3.0	5.5	6.2	2,133
10	5.0	13.5	12.7	743

Table B.1. (continued).

Reach	Length (mi)	Hydraulic Depth (ft)	Velocity (fps)	Effective Width (ft)
		25-YEAR FLOOD		
1	1.0	4.7	10.0	223
2	5.5	2.3	8.7	573
3	1.9	1.9	5.8	1,879
4	4.3	4.7	7.2	612
5	4.8	5.7	8.2	848
6	4.5	6.9	10.0	770
7	2.0	3.0	5.1	1,470
8	3.5	6.5	8.2	679
9	3.0	4.0	5.2	1,844
10	5.0	10.4	10.6	631
		10-YEAR FLOOD		
1	1.0	3.1	7.8	167
2	5.5	1.5	6.9	459
3	1.9	1.2	4.7	1,750
4	4.3	3.0	5.8	522
5	4.8	3.5	6.1	759
6	4.5	4.0	7.4	665
7	2.0	1.7	4.0	928
8	3.5	4.3	6.2	523
9	3.0	2.5	4.3	1,453
10	5.0	7.0	8.2	490
		5-YEAR FLOOD		
1	1.0	2.1	7.1	120
2	5.5	1.0	5.7	376
3	1.9	0.7	3.7	1,675
4	4.3	2.0	4.7	464
5	4.8	2.3	4.7	610
6	4.5	2.4	5.7	572
7	2.0	1.2	4.1	542
8	3.5	3.0	4.8	402
9	3.0	1.6	3.7	1,178
10	5.0	4.8	6.5	387

Table B.1. (continued).

Reach	Length (mi)	Hydraulic Depth (ft)	Velocity (fps)	Effective Width (ft)
		2-YEAR FLOOD		
1	1.0	1.2	5.6	61
2	5.5	0.5	4.3	272
3	1.9	0.4	2.5	1,570
4	4.3	1.0	3.0	353
5	4.8	1.0	2.9	463
6	4.5	1.0	3.5	443
7	2.0	1.2	4.4	214
8	3.5	1.6	3.1	242
9	3.0	0.7	2.4	830
10	5.0	2.1	4.1	292

Table B.2. Reach Averaged Hydraulic Parameters for the Main Channel of the Santa Ana River (Case 2, Postproject Conditions While Prado Reservoir is Rising).

Reach	Length (mi)	Hydraulic Depth (ft)	Velocity (fps)	Effective Width (ft)
		STANDARD PROJECT FLOOD		
1	1.0	1.5	6.2	81
2	5.5	2.7	8.2	547
3	1.9	2.3	6.6	1,940
4	4.3	6.9	8.8	710
5	4.8	11.7	10.5	1,065
6	4.5	12.9	13.9	912
7	2.0	5.8	6.7	2,234
8	3.5	10.8	11.6	881
9	3.0	7.0	7.1	2,336
10	5.0	16.5	14.2	838
		100-YEAR FLOOD		
1	1.0	1.5	6.2	81
2	5.5	2.4	7.9	523
3	1.9	2.0	6.1	1,893
4	4.3	5.5	7.7	634
5	4.8	10.0	9.7	1,028
6	4.5	11.2	13.0	879
7	2.0	5.1	6.2	2,066
8	3.5	9.7	10.8	836
9	3.0	6.2	6.6	2,232
10	5.0	14.8	13.5	764
		50-YEAR FLOOD		
1	1.0	1.5	6.2	81
2	5.5	1.9	7.2	477
3	1.9	1.6	5.4	1,825
4	4.3	4.3	6.9	579
5	4.8	6.8	8.7	910
6	4.5	8.1	11.0	805
7	2.0	3.6	5.5	1,660
8	3.5	7.5	8.9	731
9	3.0	4.6	5.6	1,976
10	5.0	11.8	11.5	681

Table B.2. (continued).

Reach	Length (mi)	Hydraulic Depth (ft)	Velocity (fps)	Effective Width (ft)
		25-YEAR FLOOD		
1	1.0	1.5	6.2	81
2	5.5	1.5	6.6	431
3	1.9	1.2	4.9	1,763
4	4.3	3.3	6.1	533
5	4.8	4.8	7.3	821
6	4.5	5.7	9.0	732
7	2.0	2.5	4.6	1,287
8	3.5	5.6	7.4	624
9	3.0	3.3	4.8	1,705
10	5.0	9.1	9.7	580
		10-YEAR FLOOD		
1	1.0	1.0	4.6	42
2	5.5	1.1	5.4	359
3	1.9	0.9	4.1	1,657
4	4.3	2.2	4.9	473
5	4.8	3.0	5.5	698
6	4.5	3.3	6.8	626
7	2.0	1.5	3.4	856
8	3.5	3.7	5.6	478
9	3.0	2.2	4.1	1,349
10	5.0	6.1	7.5	450
		5-YEAR FLOOD		
1	1.0	0.4	2.4	120
2	5.5	0.9	4.2	376
3	1.9	0.6	3.3	1,675
4	4.3	1.6	4.0	464
5	4.8	1.9	4.2	610
6	4.5	2.0	5.2	572
7	2.0	1.0	4.2	542
8	3.5	2.7	4.4	402
9	3.0	1.3	3.5	1,178
10	5.0	4.1	5.9	387

Table B.2. (continued).

Reach	Length (mi)	Hydraulic Depth (ft)	Velocity (fps)	Effective Width (ft)
		2-YEAR FLOOD		
1	1.0	0.3	3.3	61
2	5.5	0.5	2.7	272
3	1.9	1.1	3.0	1,570
4	4.3	0.8	2.4	353
5	4.8	0.9	2.8	463
6	4.5	0.9	3.0	443
7	2.0	2.1	3.5	214
8	3.5	2.2	3.9	242
9	3.0	1.6	3.6	830
10	5.0	0.3	2.9	292

Table B.3. Reach Averaged Hydraulic Parameters for the Main Channel of the Santa Ana River (Case 3, Postproject Conditions While Prado Reservoir is Falling).

Reach	Length (mi)	Hydraulic Depth (ft)	Velocity (fps)	Effective Width (ft)
STANDARD PROJECT FLOOD				
1	1.0	2.6	7.4	145
2	5.5	1.2	6.1	395
3	1.9	0.8	3.9	1,744
4	4.3	2.1	4.8	472
5	4.8	1.9	4.3	578
6	4.5	1.8	4.9	531
7	2.0	0.9	4.1	386
8	3.5	2.5	4.2	344
9	3.0	1.2	3.3	1,049
10	5.0	3.7	5.6	342
100-YEAR FLOOD				
1	1.0	2.5	7.4	135
2	5.5	1.1	5.9	383
3	1.9	0.8	4.0	1,647
4	4.3	2.0	4.7	463
5	4.8	1.8	4.1	564
6	4.5	1.7	4.7	521
7	2.0	0.8	4.1	362
8	3.5	2.4	4.1	331
9	3.0	1.1	3.2	1,018
10	5.0	3.4	5.4	333
50-YEAR FLOOD				
1	1.0	2.3	7.3	127
2	5.5	1.0	5.7	370
3	1.9	0.8	3.8	1,641
4	4.3	1.8	4.5	452
5	4.8	1.7	3.9	550
6	4.5	1.6	4.5	510
7	2.0	0.8	4.0	337
8	3.5	2.3	3.9	317
9	3.0	1.0	3.1	991
10	5.0	3.2	5.2	324

Table B.3. (continued).

Reach	Length (mi)	Hydraulic Depth (ft)	Velocity (fps)	Effective Width (ft)
		25-YEAR FLOOD		
1	1.0	1.9	6.9	113
2	5.5	0.9	5.3	344
3	1.9	0.6	3.4	1,622
4	4.3	1.6	4.1	432
5	4.8	1.4	3.6	521
6	4.5	1.3	4.1	489
7	2.0	0.6	3.9	290
8	3.5	2.0	3.6	288
9	3.0	0.9	2.8	932
10	5.0	2.8	4.8	308
		10-YEAR FLOOD		
1	1.0	1.6	6.7	88
2	5.5	0.7	4.9	304
3	1.9	0.5	2.9	1,591
4	4.3	1.3	3.5	402
5	4.8	1.2	3.2	476
6	4.5	1.1	3.7	452
7	2.0	1.2	4.5	224
8	3.5	1.7	3.2	251
9	3.0	0.7	2.7	839
10	5.0	2.3	4.2	293
		5-YEAR FLOOD		
1	1.0	1.6	6.5	77
2	5.5	0.6	4.6	276
3	1.9	0.4	2.7	1,582
4	4.3	1.1	3.2	388
5	4.8	1.0	2.9	454
6	4.5	0.9	3.5	431
7	2.0	1.1	4.3	202
8	3.5	1.5	3.0	232
9	3.0	0.6	2.3	799
10	5.0	2.0	3.9	293

Table B.3. (continued).

Reach	Length (mi)	Hydraulic Depth (ft)	Velocity (fps)	Effective Width (ft)
		2-YEAR FLOOD		
1	1.0	1.0	4.7	43
2	5.5	0.3	3.5	226
3	1.9	0.2	2.0	1,550
4	4.3	0.6	2.1	320
5	4.8	0.6	2.0	352
6	4.5	0.5	2.4	370
7	2.0	0.6	3.5	120
8	3.5	0.9	2.1	160
9	3.0	0.3	1.8	648
10	5.0	1.0	2.6	239

APPENDIX C

DEVELOPMENT OF SEDIMENT TRANSPORT RELATIONS

I. General

Transport of the bed material load in a river channel is divided into two zones. The sediment moving in a layer close to the bed is referrd to as the bed load. The sediment carried in the remaining upper region of the flow is referred to as suspended bed load. The total bed material load is the sum of the bed load and suspended bed load. The turbulent mixing process and the action of gravity on the sediment particles cause a continual transfer between the two zones. Although there is no distinct line between the zones, the definitions are made in order to aid in the mathematical description of the process. The wash load occupies the entire depth of flow but consists of fine particles that are not present in the bed in appreciable quantities, and will not easily settle out. Wash load is primarily controlled by the supply of fine silts and clays from the watershed, while bed load and suspended bed load are primarily controlled by the transport capacity of the river and availability of material in the channel bed and banks.

Sediments of different sizes will experience different rates of transport. Therefore, the transport capacities for a range of sediment sizes are determined and totaled to produce an acceptable determination of total transport capacity. The total bed material transport capacity for a channel section is

$$Q_s = T \quad P_i \,(q_{bi} + q_{si}) \qquad \text{(C.1)}$$

In Equation (C.1), T is the top width of the channel, P_i is the fraction of one sediment size, q_{bi} is the bed load transport rate per unit width for the ith size, and q_{si} is the suspended load transport rate for the ith size.

II. Bed Load Transport Capacity

The Meyer-Peter, Muller formula gives good results for bed load transport over a wide range of sediment sizes. The Meyer-Peter, Muller formula is well suited to model the dynamics of channel armoring processes as well as transport of sand sizes with little armoring potential. The formula is

$$q_{bi} = \frac{12.85}{\sqrt{\rho}\,\gamma_s} (\tau_o - \tau_c)^{1.5} \qquad \text{(C.2)}$$

in which

$$\tau_c = F_* (\gamma_s - \gamma) d_{si} \tag{C.3}$$

In Equations (C.2) and (C.3), q_{bi} is the bed load transport rate in volume per unit width for a specific size of sediment, τ_c is the critical tractive force necessary to initiate particle motion, ρ is the density of water, γ_s is the specific weight of sediment, γ is the specific weight of water, d_s is the sediment size, and F_* is the Shield's parameter which ranges between 0.030 and 0.060 inclusive.

The boundary shear stress acting on the grain is

$$\tau_o = \frac{f_o}{8} \rho V^2 \tag{C.4}$$

where ρ is the density of flowing water and f_o is the Darcy-Weisbach friction factor, and V is the mean velocity of the flow.

The Darcy-Weisbach friction factor is related to Manning's n by:

$$f_o = n^2 \frac{8g}{1.49^2 R^{1/3}} \tag{C.5}$$

where g is the acceleration due to gravity and R is hydraulic radius. Assuming the wide channel approximation is valid then the hydraulic radius R is equal to depth.

For sediment transport calculations, Manning's n in Equation C.5 represents the skin resistance only, which is typically 0.6 to 0.9 times the total Manning's n for the channel depending upon the preponderance and type of bed forms.

III. Suspended Bed Material Transport Capacity

The suspended sediment transport capacity is determined by using a solution developed by Einstein. This method relies upon integration of the sediment concentration profile as a function of depth. The nature of the profile is determined using turbulent transport theory. The sediment profile is assumed to be in equilibrium, and therefore the rate at which sediment is transported upward due to turbulence and the concentration gradient is exactly equal to the rate at which gravity is transporting sediment downward. The resulting concentration profile is given by:

$$\frac{C}{C_a} = \left[\frac{Y-y}{y}\frac{a}{Y-a}\right]^z \tag{C.6}$$

where C is the concentration at a point y, C_a is the concentration at point a generally considered to be the bed layer thickness, Y is the depth and z is given by:

$$z = \frac{w}{\kappa U_*} \tag{C.7}$$

where w is the fall velocity for a given size sediment, U_* is the shear velocity, and κ is the von Karman constant (0.4). U_* is defined as $\sqrt{\tau/\rho}$ where τ is the shear stress on the bed and ρ is the density of the fluid. (Note: τ should be the total shear stress on the bed and not the grain-associated shear stress. The shear velocity U_* characterizes both the turbulence and grain resistance, and the fall velocity, w, characterizes the sediment properties.)

A logarithmic velocity profile is used to describe the velocity distribution in turbulent flows. The equation utilized is

$$\frac{u_\xi}{U_*} = B + 2.5\ \ell n\left(\frac{\xi}{\eta_s}\right) \tag{C.8}$$

in which u_ξ is the point mean velocity at the distance ξ from the bed, B is a constant dependent on roughness, and η_s is the roughness height.

The integral of suspended load above the bed layer is obtained by combining Equations C.6 and C.7 or

$$q_{si} = \int_a^Y u_\xi C_\xi d\xi = C_a U_* \int_a^Y \left[B + 2.5\ \ell n\left(\frac{\xi}{\eta_s}\right)\right]\left(\frac{Y-\xi}{\xi}\frac{a}{Y-a}\right)^z d\xi \tag{C.9}$$

where q_{si} is the suspended bed material transport for a specific size sediment above the bed layer. In this equation

$$\sigma = \frac{\xi}{Y} \tag{C.10}$$

where σ is the dimensionless relative depth and

$$G = \frac{a}{Y}$$

in which G is the relative depth of the bed layer. Integration of Equation C.9 yields

$$q_{si} = C_a U_* a \frac{G^{z-1}}{(1-G)^z} \{[B + 2.5\ \ell n(\frac{Y}{\eta_s})] \int_G^1 (\frac{1-\sigma}{\sigma})^z d\sigma + 2.5 \int_G^1 (\frac{1-s}{\sigma})^z \ell n\sigma\, d\sigma\} \tag{C.11}$$

The average flow velocity V is defined by the equation

$$V = \frac{\int_0^Y u_\xi d\xi}{\int_0^Y d\xi} \tag{C.12}$$

Substituting Equation C.8 into Equation C.12 and integrating yields

$$\frac{V}{U_*} = B + 2.5\ \ell n(\frac{Y}{\eta_s}) - 2.5 \tag{C.13}$$

The two integrals in Equation C.11 can be defined as

$$J_1 = \int_G^1 (\frac{1-\sigma}{\sigma})^z\, d\sigma \tag{C.14}$$

and

$$J_2 = \int_G^1 (\frac{1-\sigma}{\sigma})^z\ \ell n\sigma\, d\sigma \tag{C.15}$$

Substituting Equations C.13, C.14, and C.15 reduces Equation C.11 to

$$q_{si} = C_a U_* a \frac{G^{z-1}}{1-G^z} [(\frac{V}{U_*} + 2.5) J_1 + 2.5 J_2] \tag{C.16}$$

According to Einstein, the concentration at the upper level of the bed layer is related to the total transport in the bed layer by .

$$q_{bi} = 11.6\ C_a U_* a \tag{C.17}$$

Substituting into Equation C.16 produces

$$q_{si} = \frac{q_{bi}}{11.6} \frac{G^{z-1}}{1-G^z} [(\frac{V}{U_*} + 2.5) J_1 + 2.5 J_2] \tag{C.18}$$

Generally sediment measurements are taken in a zone approximately 0.3 feet above the bed and higher since commonly used measuring devices are incapable of getting closer to the bed. Hence, commonly reported sediment measurements consist of wash load and suspended bed load. This zone of measured sediment transport will be denoted the "measured zone" in the following discussion.

For practical purposes wash load consists of no particles greater than 0.062 mm. If size analyses of suspended sediment and corresponding concentration measurements have been made, then the concentration of suspended bed material in the measured zone is given by

$$C_{bm} = P * C_m \tag{C.19}$$

where C_{bm} is the concentration of suspended bed material in the measured zone, P is percent of suspended bed material load and C_m is total measured concentration.

Now C_{bm} may be calculated theoretically using the following procedure. First the suspended bed-material concentration at 0.3 feet above the bed is calculated from:

$$C_{0.3} = C_a \left(\frac{Y - 0.3}{0.3} \frac{a}{Y - a}\right)^z \tag{C.20}$$

Substituting this concentration into Equation C.16 and changing the limits of integration appropriately yields the transport rate of suspended bed material for a specific size in the zone extending from 0.3 feet above the bed to the water surface. The resulting relation is

$$q_{0.3i} = C_{0.3}\, U_* 0.3 \frac{G'^{z-1}}{(1-G')^z} \left[\left(\frac{V}{U_*} + 2.5\right) J_1' + 2.5\, J_2'\right] \tag{C.21}$$

where

$$G' = \frac{0.3}{Y} \tag{C.22}$$

$$J_1' = \int_{G'}^{1} \left(\frac{1-\sigma}{\sigma}\right)^z d\sigma \tag{C.23}$$

and

$$J_2' = \int_{G'}^{1} \left(\frac{1-\sigma}{\sigma}\right)^z \ln\sigma \, d\sigma \tag{C.24}$$

Rewriting Equation C.21 and using Equation C.20 produces the relation

$$q_{0.3i} = C_a U_* 0.3 \left(\frac{Y-0.3}{0.3} \frac{a}{y-a}\right)^z \frac{G'^{z-1}}{(1-G')^z} \left[\left(\frac{V}{U_*} + 2.5\right) J_1' + 2.5 J_2'\right] \tag{C.25}$$

Introducing Equation C.17 modifies the foregoing relation to read

$$q_{0.3i} = \frac{q_{bi}}{11.6} \frac{0.3}{a} \left(\frac{Y-0.3}{0.3}\right)^z \frac{G'^{z-1}}{(1-G')^1} \left[\left(\frac{V}{U_*}\right) + 2.5\right) J_1' + 2.5 J_2'\right] \tag{C.26}$$

The total concentration of suspended bed material in the measured zone is given by

$$C'_{bm} = \sum_{i=1}^{n} (P_i * q_{0.3i}) / q * S.G. \tag{C.27}$$

where C'_{bm} is the theoretically computed bed-material concentration, q is water discharge, and S.G. is the specific gravity of the sediment. n is the total number of sediment sizes.

Comparing C_{bm} and C'_{bm} for a wide range of discharges yields a measure of the accuracy of the theoretical computation procedure.

This procedure can be "calibrated" so that the C'_{bm} values agree closely with the C_{bm} values by adjusting one or more parameters involved in the procedure. These parameters are:

1. Shield's parameter F_*. This is generally assumed equal to 0.047 but may range in value from 0.030 to approximately 0.060.

2. Manning's n for skin resistance only. Typical values are 0.01 to 0.03. This parameter may be estimated from Stickler's relation:

$$n = \frac{D_{90}^{1/6}}{26}$$

where D_{90} is in meters.

3. Bed layer thickness a. This is an extremely important parameter since it determines the concentration C_a which defines the suspended bed material concentration profile. The value of a is typically taken as two times the D_{65} or D_{90} of the bed material.

4. As a last resort the exponent 1.5 and the coefficient 12.85 in Equation C.2 may be adjusted.

The next step is to use the "calibrated" transport procedure to generate values for total bed material transport for a wide range of river discharges. HEC-2 was used to determine the hydraulics for these discharges. A regression analysis was performed on the resulting data to obtain a relation of the form:

$$q_s = a\ V^b\ Y^c \qquad (C.28)$$

where q_s is unit bed material transport, v is velocity, Y is depth, and a, b and c are coefficients.

The calibrated transport relations are then applied using the average hydraulic and sediment conditions in each of the computational reaches to determine the bed-material rating curves.

$$Q_s = a\ Q^b \qquad (C.29)$$

for each reach, where Q_s is total bed material transport within the reach and Q is river discharge. These relationships form the basis of the continuity calculations.

APPENDIX D

SEDIMENT TRANSPORT AND AGGRADATION/ DEGRADATION ANALYSES RESULTS

Table D.1. Sediment Transport and Aggradation/Degradation Summary for Case 1 (Preproject Conditions).

Reach	Flood Event	Instantaneous Peak Sediment Transport Capacity (cfs)	Volumetric Sediment Transport Capacity (10^{-3} YD^3)	Net Agg/Deg Volume (x10^{-3} YD^3)	Bulked Net Agg/Deg Volume (x10^{-3} YD^3)	Bulked Net Agg/Deg Depth (ft)
1	SPF	137	291	--	--	--
	100-yr	109	232	--	--	--
	50-yr	62	132	--	--	--
	25-yr	35	73	--	--	--
	10-yr	14	29	--	--	--
	5-yr	6	12	--	--	--
	2-yr	1	2	--	--	--
2	SPF	478	1,001	-710	-994	-1.07
	100-yr	333	697	-465	-651	-0.77
	50-yr	179	375	-243	-340	-0.47
	25-yr	97	202	-129	-181	-0.29
	10-yr	38	80	- 51	- 71	-0.14
	5-yr	16	34	- 22	- 31	-0.08
	2-yr	3	7	- 5	- 7	-0.02
3	SPF	175	376	+625	+875	+1.10
	100-yr	123	265	432	605	0.78
	50-yr	67	145	230	322	0.44
	25-yr	37	79	123	172	0.25
	10-yr	15	32	48	67	0.10
	5-yr	6	14	20	28	0.04
	2-yr	1	3	4	6	0.01
4	SPF	144	275	+101	+141	+0.20
	100-yr	89	171	94	132	0.20
	50-yr	46	89	56	78	0.14
	25-yr	24	46	33	46	0.09
	10-yr	9	17	15	21	0.05
	5-yr	4	7	7	10	0.03
	2-yr	1	1	2	3	0.01
5	SPF	248	460	-185	-259	-0.24
	100-yr	189	351	-180	-252	-0.25
	50-yr	105	194	-105	-147	-0.16
	25-yr	51	94	- 48	- 67	-0.08
	10-yr	17	31	- 14	- 20	-0.03
	5-yr	6	11	- 4	- 6	-0.01
	2-yr	1	2	- 1	- 1	-0.00
6	SPF	38	70	+390	+546	+0.63
	100-yr	29	54	297	416	0.51
	50-yr	16	30	164	230	0.31
	25-yr	8	15	79	111	0.16
	10-yr	3	5	76	36	0.06
	5-yr	1	2	9	13	0.03
	2-yr	0	0	2	3	0.01
7	SPF	125	333	-263	-368	-0.37
	100-yr	103	276	-222	-311	-0.35
	50-yr	67	180	-150	-210	-0.28
	25-yr	40	106	- 91	-127	-0.22
	10-yr	18	48	- 43	- 60	-0.17
	5-yr	9	23	- 21	- 29	-0.14
	2-yr	2	6	- 6	- 8	-0.10

Table D.1. (continued).

Reach	Flood Event	Instantaneous Peak Sediment Transport Capacity (cfs)	Volumetric Sediment Transport Capacity (10^{-3} YD^3)	Net Agg/Deg Volume ($x10^{-3}$ YD^3)	Bulked Net Agg/Deg Volume ($x10^{-3}$ YD^3)	Bulked Net Agg/Deg Depth (ft)
8	SPF	511	841	-508	-711	-1.10
	100-yr	373	614	-338	-473	-0.77
	50-yr	183	300	-120	-168	-0.31
	25-yr	75	124	- 18	- 25	-0.05
	10-yr	20	33	+ 15	+ 21	+0.06
	5-yr	6	10	+ 13	+ 18	+0.07
	2-yr	1	1	+ 5	+ 7	+0.04
9	SPF	179	342	+499	+699	+0.48
	100-yr	137	262	352	493	0.36
	50-yr	75	143	157	220	0.18
	25-yr	35	67	57	80	0.07
	10-yr	11	22	11	15	0.02
	5-yr	4	8	2	3	0.00
	2-yr	1	1	0	0	0.00
10	SPF	121	244	+ 98	+137	+0.15
	100-yr	94	189	73	102	0.12
	50-yr	53	107	36	50	0.07
	25-yr	26	52	15	21	0.03
	10-yr	9	18	4	6	0.01
	5-yr	3	7	1	1	0.00
	2-yr	1	1	0	0	0.00

Table D.2. Sediment Transport and Aggradation/Degradation Summary for Case 2 (Postproject Conditions While Prado Reservoir Is Rising).

Reach	Flood Event	Instantaneous Peak Sediment Transport Capacity (cfs)	Volumetric Sediment Transport Capacity (10^{-3} YD^3)	Net Agg/Deg Volume ($\times 10^{-3}$ YD^3)	Bulked Net Agg/Deg Volume ($\times 10^{-3}$ YD^3)	Bulked Net Agg/Deg Depth (ft)
1	SPF	2	30 (2)*	- 30	- 42	-2.65
	100-yr	2	30 (2)	- 30	- 42	-2.65
	50-yr	2	30 (2)	- 30	- 42	-2.65
	25-yr	2	30 (2)	- 30	- 42	-2.65
	10-yr	1	6 (0)	- 6	- 8	-0.97
	5-yr	0	0	0	0	0.00
	2-yr	0	0	0	0	0.00
2	SPF	145	360	-358	-501	-0.85
	100-yr	109	269	-267	-374	-0.66
	50-yr	69	171	-169	-237	-0.46
	25-yr	42	105	-103	-144	-0.31
	10-yr	19	48	- 48	- 67	-0.17
	5-yr	9	23	- 23	- 32	-0.10
	2-yr	2	6	- 6	- 8	-0.03
3	SPF	55	140	+220	+308	+0.43
	100-yr	41	105	164	230	0.33
	50-yr	27	68	103	144	0.21
	25-yr	16	42	63	88	0.13
	10-yr	8	20	28	39	0.06
	5-yr	4	10	13	18	0.03
	2-yr	1	3	3	4	0.01
4	SPF	56	112	+ 28	+ 39	+0.07
	100-yr	28	57	48	67	0.13
	50-yr	17	34	34	48	0.10
	25-yr	10	19	23	32	0.07
	10-yr	4	8	12	17	0.04
	5-yr	2	4	6	8	0.02
	2-yr	1	1	2	3	0.01
5	SPF	175	324	-212	-297	-0.30
	100-yr	133	246	-189	-265	-0.27
	50-yr	70	130	- 96	-134	-0.16
	25-yr	35	64	- 45	- 63	-0.08
	10-yr	12	22	- 14	- 20	-0.03
	5-yr	4	8	- 4	- 6	-0.01
	2-yr	1	1	- 0	0	0.00
6	SPF	27	50	+274	+384	+0.48
	100-yr	20	38	208	291	0.38
	50-yr	11	20	110	154	0.22
	25-yr	5	10	54	76	0.12
	10-yr	2	3	19	27	0.05
	5-yr	1	1	7	10	0.02
	2-yr	0	0	1	1	0.00
7	SPF	98	262	-212	-297	-0.34
	100-yr	80	216	-178	-249	-0.31
	50-yr	50	136	-116	-162	-0.25
	25-yr	30	81	- 71	- 99	-0.20
	10-yr	14	37	- 34	- 48	-0.14
	5-yr	6	17	- 16	- 22	-0.13
	2-yr	2	4	- 4	- 6	-0.08

Table D.2. (continued).

Reach	Flood Event	Instantaneous Peak Sediment Transport Capacity (cfs)	Volumetric Sediment Transport Capacity $(10^{-3}\ YD^{3})$	Net Agg/Deg Volume $(x10^{-3}\ YD^{3})$	Bulked Net Agg/Deg Volume $(x10^{-3}\ YD^{3})$	Bulked Net Agg/Deg Depth (ft)
8	SPF	340	550	-288	-403	-0.67
	100-yr	246	399	-183	-256	-0.45
	50-yr	113	183	- 47	- 66	-0.13
	25-yr	48	78	+ 3	+ 4	+0.01
	10-yr	13	21	+ 16	+ 22	+0.07
	5-yr	4	6	+ 11	+ 15	+0.06
	2-yr	0	1	+ 3	+ 4	+0.03
9	SPF	126	240	+310	+434	+0.32
	100-yr	96	183	216	302	0.23
	50-yr	50	94	89	125	0.11
	25-yr	24	46	32	45	0.04
	10-yr	8	15	6	8	0.01
	5-yr	3	5	1	1	0.00
	2-yr	0	1	0	0	0.00
10	SPF	87	175	+ 65	+ 91	+0.11
	100-yr	67	135	48	67	0.09
	50-yr	36	72	22	31	0.05
	25-yr	18	37	9	13	0.02
	10-yr	6	13	2	3	0.01
	5-yr	2	5	0	0	0.00
	2-yr	0	1	0	0	0.00

*Actual Volume of Sediment Transported as Controlled by Armoring.

Table D.3. Sediment Transport and Aggradation/Degradation Summary for Case 3 (Postproject Conditions While Prado Reservoir is Falling).

Reach	Flood Event	Instantaneous Peak Sediment Transport Capacity (cfs)	Volumetric Sediment Transport Capacity (10^{-3} YD^3)	Net Agg/Deg Volume ($\times 10^{-3}$ YD^3)	Bulked Net Agg/Deg Volume ($\times 10^{-3}$ YD^3)	Bulked Net Agg/Deg Depth (ft)
1	SPF	9	198 (8)*	-198	-277	-9.77
	100-yr	8	88 (8)	- 88	-123	-4.66
	50-yr	7	60 (7)	- 60	- 84	-3.38
	25-yr	5	25 (5)	- 25	- 35	-1.58
	10-yr	3	12 (4)	- 12	- 17	-0.99
	5-yr	2	8 (3)	- 8	- 11	-0.73
	2-yr	1	2 (0)	- 2	- 3	-0.36
2	SPF	20	422 (179)	-414	-580	-1.37
	100-yr	17	188 (156)	-180	-252	-0.61
	50-yr	14	127	-120	-168	-0.42
	25-yr	10	53	- 48	- 67	-0.18
	10-yr	6	25	- 21	- 29	-0.09
	5-yr	5	16	- 13	- 18	-0.06
	2-yr	1	3	- 3	- 4	-0.02
3	SPF	8	169	+ 10	+ 14	+0.02
	100-yr	7	75	81	113	0.18
	50-yr	6	51	76	106	0.17
	25-yr	4	22	31	43	0.07
	10-yr	2	11	14	20	0.03
	5-yr	2	7	9	13	0.02
	2-yr	0	2	1	1	0.00
4	SPF	4	90	+ 79	+111	+0.28
	100-yr	4	40	35	49	0.13
	50-yr	3	26	25	35	0.09
	25-yr	2	11	11	15	0.04
	10-yr	1	5	6	8	0.02
	5-yr	1	3	4	6	0.02
	2-yr	0	1	1	1	0.00
5	SPF	4	75	+ 15	+ 21	+0.04
	100-yr	3	34	6	8	0.02
	50-yr	3	22	4	6	0.01
	25-yr	2	9	2	3	0.01
	10-yr	1	4	1	1	0.00
	5-yr	1	3	0	0	0.00
	2-yr	0	1	0	0	0.00
6	SPF	1	12	+ 63	+ 88	+0.19
	100-yr	1	5	29	41	0.09
	50-yr	0	4	18	25	0.06
	25-yr	0	1	8	11	0.03
	10-yr	0	1	3	4	0.01
	5-yr	0	0	3	4	0.01
	2-yr	0	0	1	1	0.00
7	SPF	6	131	-119	-167	-1.11
	100-yr	5	57	- 52	- 73	-0.52
	50-yr	4	42	- 38	- 53	-0.40
	25-yr	3	21	- 20	- 28	-0.25
	10-yr	2	12	- 11	- 15	-0.17
	5-yr	2	8	- 8	- 11	-0.14
	2-yr	1	3	- 3	- 4	-0.09

Table D.3. (continued).

Reach	Flood Event	Instantaneous Peak Sediment Transport Capacity (cfs)	Volumetric Sediment Transport Capacity (10^{-3} YD^3)	Net Agg/Deg Volume (x10^{-3} YD^3)	Bulked Net Agg/Deg Volume (x10^{-3} YD^3)	Bulked Net Agg/Deg Depth (ft)
8	SPF	3	56	+ 75	+105	+0.45
	100-yr	2	25	32	45	0.20
	50-yr	2	15	27	38	0.18
	25-yr	1	5	16	22	0.11
	10-yr	1	2	10	14	0.08
	5-yr	0	1	7	10	0.06
	2-yr	0	0	3	4	0.04
9	SPF	2	45	+ 11	+ 15	+0.02
	100-yr	2	20	5	7	0.01
	50-yr	2	13	2	3	0.01
	25-yr	1	5	0	0	0.00
	10-yr	1	2	0	0	0.00
	5-yr	0	1	0	0	0.00
	2-yr	0	0	0	0	0.00
10	SPF	2	40	+ 5	+ 7	+0.02
	100-yr	2	18	2	3	0.01
	50-yr	1	12	1	1	0.00
	25-yr	1	5	0	0	0.00
	10-yr	1	2	0	0	0.00
	5-yr	0	1	0	0	0.00
	2-yr	0	0	0	0	0.00

*Actual Volume of Sediment Transported as Controlled by Armoring.

APPENDIX E

SCOUR DEPTH AND SCOUR HOLE CONFIGURATION

Scour depths for both Alternative 2, and the flip bucket alternative were computed using the method developed by Chee & Yuen (1985). This method was selected because it was developed for conditions similar to those at the Seven Oaks Dam site. The method was developed based on model and prototype data where coarse bed material was present; it accounts for tailwater effects and the angle at which the jet impacts the tailwater. This method was chosen over the traditional Veronese equation (1937) because this equation applies to vertical jets only and was developed for relatively fine bed material.

The Chee & Yuen method computes the scour depth associated with a vertical jet using

$$H_s = \frac{0.558\ U_o\ Y_o^{0.5}}{d_m^{0.1}} \qquad \text{(E.1)}$$

where H_s = Vertical jet scour depth (M)

U_o = Velocity of jet at impact (M/S)

Y_o = Depth of flow of jet at impact (M)

d_m = Mean particle diameter; d_{50} (M)

The vertical jet scour depth is then adjusted for the angle of the jet at impact and tailwater as follows:

$$D_{m\infty} = H_s \text{ SIN } \beta - Y_{TW} \qquad \text{(E.2)}$$

where $D_{m\infty}$ = Angled jet scour depth (M)

β = Angle of jet at impact

Y_{TW} = Tailwater depth (M)

The vertical and angled jet scour depths were computed for both Alternative 2 and the flip bucket alternative for a range of discharges, including the design discharge (8,000 cfs) and the conduit capacity discharge (15,000 cfs). The results are given in Table E.1. These depths were computed assuming the mean subsurface particle diameter is 15 mm and neglecting tailwater. Data on the size of subsurface material were not available so the surface material gradation curve (d_{50} = 90 mm) was adjusted to reflect a reduction in particle size with depth; the resulting d_{50} was 15 mm (see Figure E.1). Due to the uncertainty of tailwater depths in the impact area, the conservative assumption of no tailwater was made. Due to the relatively

Table E.1. Scour Depths as Computed Using the Chee & Yuen Equation (Neglecting Tailwater).

Discharge (cfs)	Scour Depth (ft) Alternative 2 Vertical Jet	Scour Depth (ft) Alternative 2 Angled Jet	Scour Depth (ft) Flip Bucket Alternative Vertical Jet	Scour Depth (ft) Flip Bucket Alternative Angled Jet
500	13	10	10	9
2,000	37	20	22	17
4,000	--	--	30	21
8,000	105	34	38	26
15,000	154	48	43	28

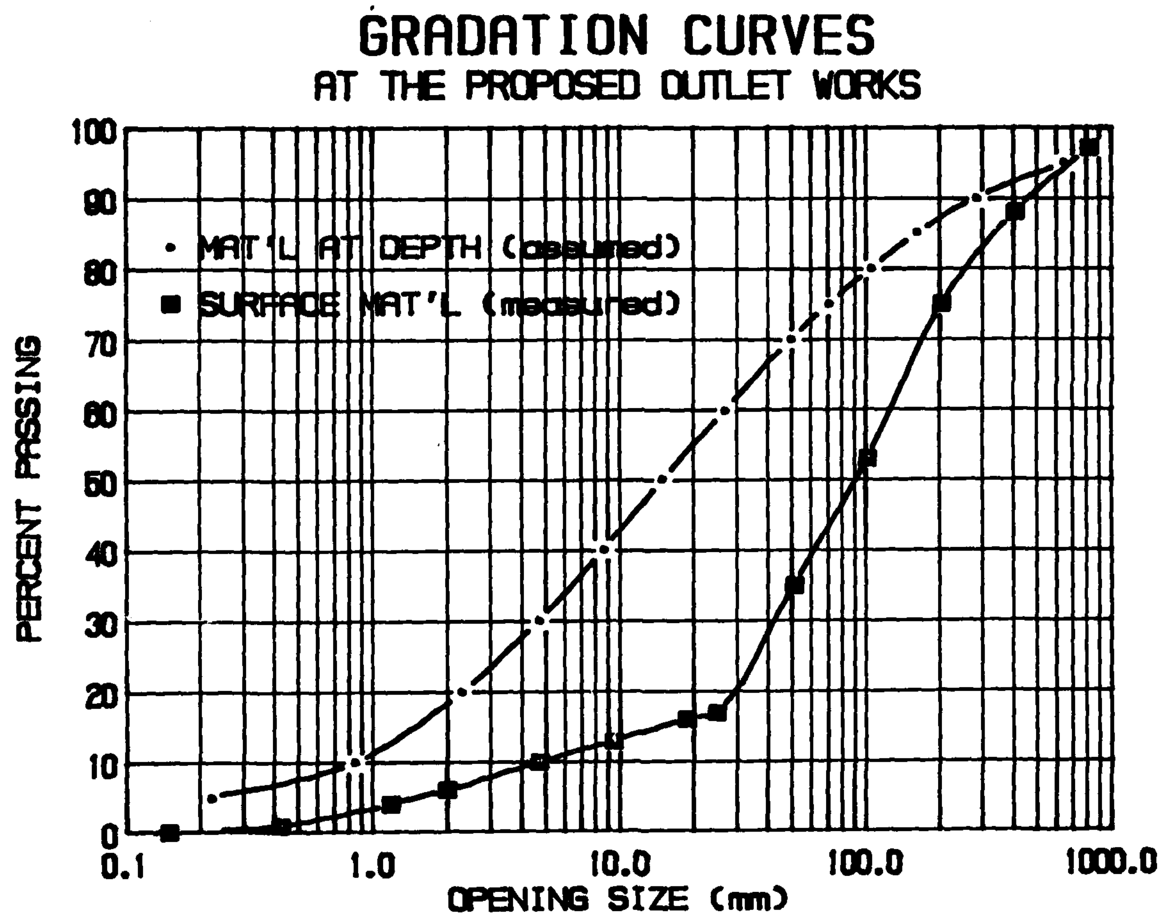


Figure E.1. Gradation curves in the vicinity of the proposed outlet works.

flat trajectory and short distance traveled by the Alternative 2 jet, dispersion of the jet was neglected. For the flip bucket alternative, the jet was assumed to disperse as prescribed by the Bureau of Reclamation (1978). The angle of dispersion of one side of the jet is a function of Froude number and flip bucket angle and ranged from 2.2 to 4.0 degrees for the discharges analyzed.

The Chee & Yuen vertical jet scour depths were compared to the scour depths computed using the Veronese equation. The results are given in Table E.2. It can be seen that the two methods are in satisfactory agreement for Alternative 2, but differ somewhat for the flip bucket alternative. Since the two methods do not depend equally on the jet velocity, the discrepancies in the flip bucket scour depths arise from the change in velocity caused by the dispersion of the jet. Because the Chee & Yuen equations were developed using larger diameter particles than the Veronese equation, it would be expected that the Chee & Yuen method would yield smaller scour depths than the Veronese equation.

The sensitivity of the Chee & Yuen method to mean particle diameter was investigated by computing scour depths for a range of particle sizes. As can be seen in Table E.3, scour depth is relatively insensitive to particle size. This result was expected as the exponent on d_m in Equation E.1 is only 0.1.

The sensitivity of scour depth with impact angle (measured from vertical axis) was analyzed because Chee & Yuen investigated impact angles between 45 and 90 degrees only, and several of the scenarios currently being investigated have impact angles less than 45 degrees. The results of this analysis are given in Table E.4. It was found that assuming a minimum impact angle of 45 degrees has a substantial effect on scour depth when the actual impact angle is significantly less than 45 degrees.

Although the Chee & Yuen method is felt to be the best method available for computing scour depths, uncertainty in estimating scour depths remains a problem. Therefore, the Corps of Engineers (COE) requested that a reasonable factor of safety be determined and used to compute maximum possible scour depths. It was decided that the Chee & Yuen method should not be extrapolated beyond the limits for which it was developed; so the impact angle was limited to a minimum of 45 degrees. At the design discharge for Alternative 2, the scour depth for an impact angle of 45 degrees is approximately twice the scour depth for the actual angle. Therefore, for Alternative 2, a factor of safety

Table E.2. Comparison of Scour Depths Computed Using the Chee & Yuen and Veronese Equations.

Discharge (cfs)	Scour Depth (ft) - Based on Vertical Jet					
	Alternative 2		Percent Difference From Veronese	Flip Bucket Alternative		Percent Difference From Veronese
	Chee & Yuen	Veronese		Chee & Yuen	Veronese	
500	13	17	-24	10	15	-33
2,000	37	44	-16	20	29	-31
8,000	105	120	-13	38	52	-27

Table E.3. Sensitivity Analysis of Scour Depth and Particle Size.

Discharge (cfs)	Scour Depth (ft) - Chee & Yuen					
	Alternative 2			Flip Bucket Alternative		
	d_{50} = 2MM	d_{50} = 15MM	d_{50} = 90MM	d_{50} = 2MM	d_{50} = 15MM	d_{50} = 90MM
500	12	10	9	11	9	8
2,000	24	20	17	21	17	14
8,000	42	34	29	31	26	22
15,000	58	48	40	34	28	24

Table E.4. Sensitivity Analysis of Scour Depth and Jet Impact Angle.

Discharge (cfs)	Scour Depth (ft) - Chee & Yuen: Alternative 2		Scour Depth (ft) - Chee & Yuen: Flip Bucket Alternative	
	β_{actual}	$\beta_{min} = 45°$	β_{actual}	$\beta_{min} = 45°$
500	10*	10	9*	9
2,000	20	26	17*	17
4,000	--	--	21*	21
8,000	34	74	26	27
15,000	48	109	28	30

*$\beta_{actual} \geq \beta_{min}$

of 2.0 was applied to the "best estimate" scour depths to give the "maximum" scour depths. These depths are given in Table E.5. Because the flip bucket propels the jet away from the outlet channel, a smaller factor of safety was used for the flip bucket alternative than for the do nothing alternative. It was decided that the vertical jet scour depths would provide accurate estimates for the "maximum" scour depths for the flip bucket alternative. These depths are given in Table E.5.

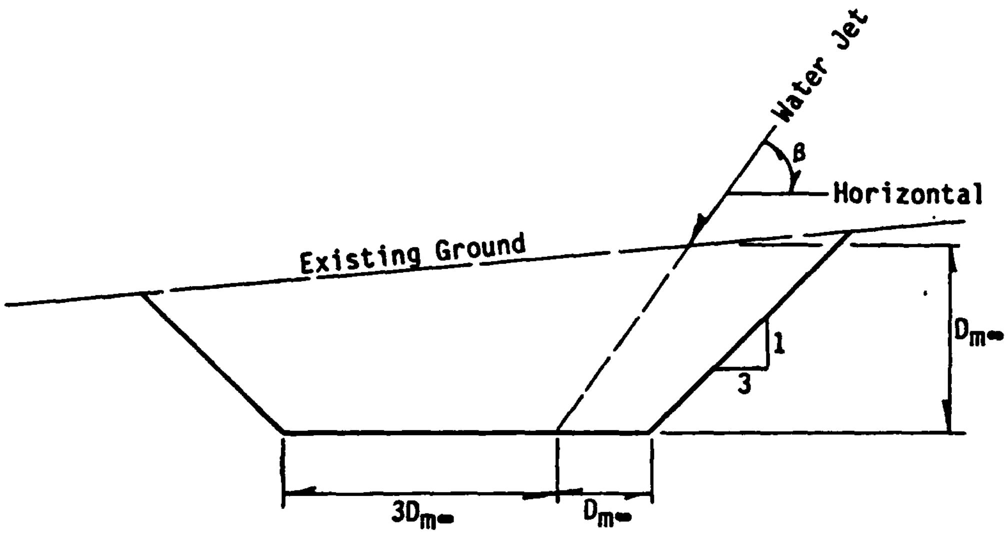
While enough confidence is placed in the best estimate scour depths to use them to complete the preliminary design, it must be stressed that considerable uncertainty exists in any method that estimates scour depths. Because of this uncertainty, maximum scour depths are provided as probable upper limits on the depths of scour. Implementation of any alternative utilizing a scour hole should not be carried out without a carefully conducted model study to provide more reliable scour depth estimates.

The typical scour hole configuration is shown in Figure E.2. Chee & Yuen found that the jet maintained its impact angle as it traveled to the bottom of the scour hole (see profile). Based on experience with physical modeling, it was assumed that the length and width of the bottom of the scour hole are equal to four times the scour depth and jet width, respectively. The length of the bottom of the scour hole is apportioned relative to the jet as shown in Figure E.2. It was conservatively estimated that all sideslopes would be 3H:1V.

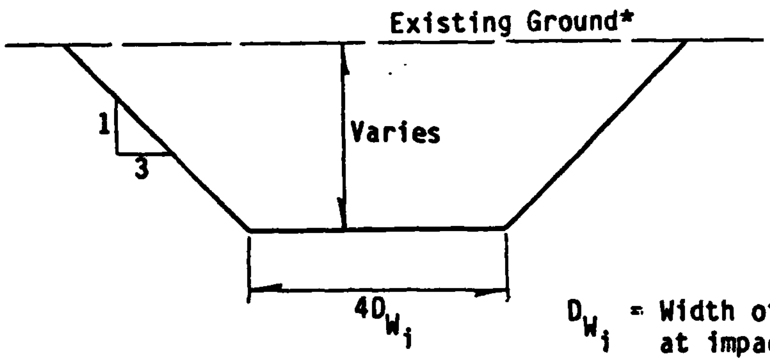
Table E.5. Summary of "Best-Estimate" and "Maximum" Scour Depths.

Discharge (cfs)	Scour Depth (ft) - Chee & Yuen			
	Alternative 2		Flip Bucket Alternative	
	Best Estimate	Maximum*	Best Estimate	Maximum**
500	10	20	9	10
2,000	20	40	17	22
4,000	--	--	21	30
8,000	34	68	26	38
15,000	48	96	28	43

*Assumes Factor of Safety of 2.0
**Assumes Vertical Jet



PROFILE



SECTION

D_{W_i} = Width of water jet at impact

*Assume existing ground along ℄ is constant for width of scour hole.

Figure E.2. Profile and section view of typical scour hole.

APPENDIX F

COST ESTIMATES

Table F.1. Preliminary Cost Estimate for the Stilling Basin Alternative.

Description	Unit	Quantity	Unit Price	Cost
Excavation	CY	34,220	$ 3.00	$ 102,700
Backfill	CY	19,110	1.50	28,700
Reinforced Concrete - Basin, Apron & Walls	CY	3,240	180.00	583,200
Reinforced Concrete - End Sill	CY	14	200.00	2,800
Reinforced Concrete - Small Dentates	--	15	100.00	1,500
Fencing	LF	970	7.00	6,800
			Net Cost	$ 725,700
			Contingency (20%)	144,300
			TOTAL COST	$ 870,000

Table F.2. Preliminary Cost Estimate for the Submerged Bucket Alternative.

Description	Unit	Quantity	Unit Price	Cost
Excavation	CY	144,200	$ 3.00	$ 432,600
Backfill	CY	20,500	1.50	30,800
Reinforced Concrete - Chute & Walls	CY	910	180.00	163,800
Reinforced Concrete - Bucket & End Structure	CY	420	200.00	84,000
Fencing	LF	1,620	7.00	11,400
			Net Cost	$ 722,600
			Contingency (20%)	144,400
			TOTAL COST	$ 867,000

Table F.3. Preliminary Cost Estimate for the Channelization Alternative.

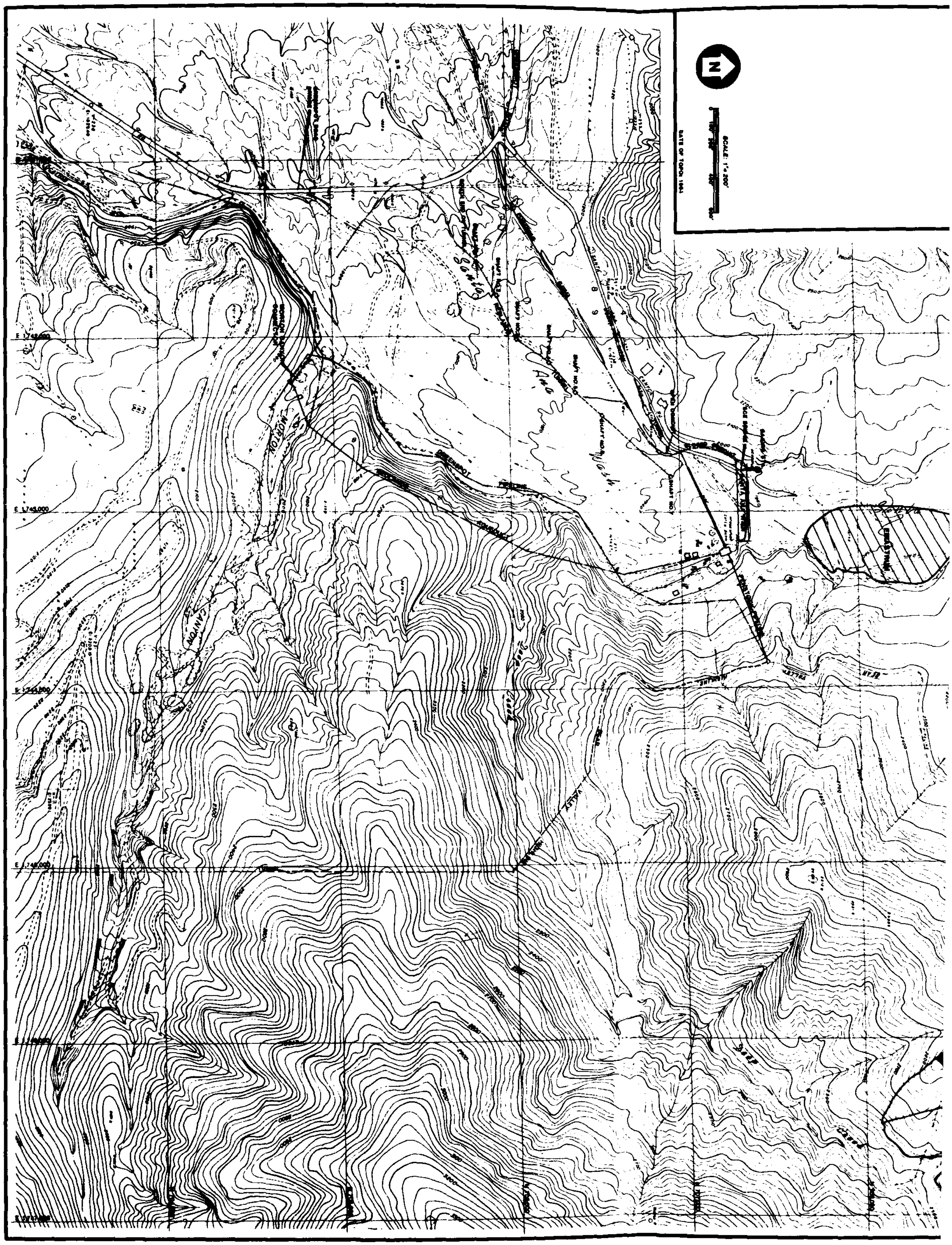
Description	Unit	Quantity	Unit Price	Cost
Excavation	CY	20,410	$ 3.00	$ 61,300
Backfill	CY	14,430	1.50	21,700
Fencing	LF	4,000	7.00	28,000
			Net Cost	$ 111,000
			Contingency (20%)	22,000
			TOTAL COST	$ 133,000

Table F.4. Preliminary Cost Estimate for the Flip Bucket with Preformed Plunge Pool and Toe Protection for the Outlet Portal Exit Channel Alternative.

Description	Unit	Quantity	Unit Price	Cost
Excavation	CY	76,300	$ 3.00	$ 228,900
Reinforced Concrete - Chute, Walls & Bucket	CY	830	180.00	149,400
Fencing	LF	1,200	7.00	8,400
			Net Cost	$ 386,700
			Contingency (20%)	77,300
			TOTAL COST	$ 464,000

Table F.5. Preliminary Cost Estimate for the Preformed Plunge Pool With Toe Protection for the Outlet Portal Exit Channel Alternative.

Description	Unit	Quantity	Unit Price	Cost
Excavation	CY	42,700	$ 3.00	$ 128,100
Reinforced Concrete - Cutoff Wall	CY	710	180.00	127,800
Fencing	LF	900	7.00	6,300
			Net Cost	$ 262,200
			Contingency (20%)	52,800
			TOTAL COST	$ 315,000



N
SCALE: 1"= 200'
DATE OF TOPO. 1964
MORTON
CANYON
E 1,745,000

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Newport Beach, CA · Phoenix, AZ · Tucson, AZ

PROJECT NO.	CA-GHS-17
DATE:	4/87
DESIGN:	GJK
DRAWN:	TBS
CHECK:	EAM
REVISIONS:	

SHEET 1

SEE TEXT PAGES 2.1 & 2.2

SANTA ANA RIVER, FLOOD CONTROL PROJECT, SEVEN OAKS DAM

LOS ANGELES DISTRICT, CORPS OF ENGINEERS

LOCATION MAP

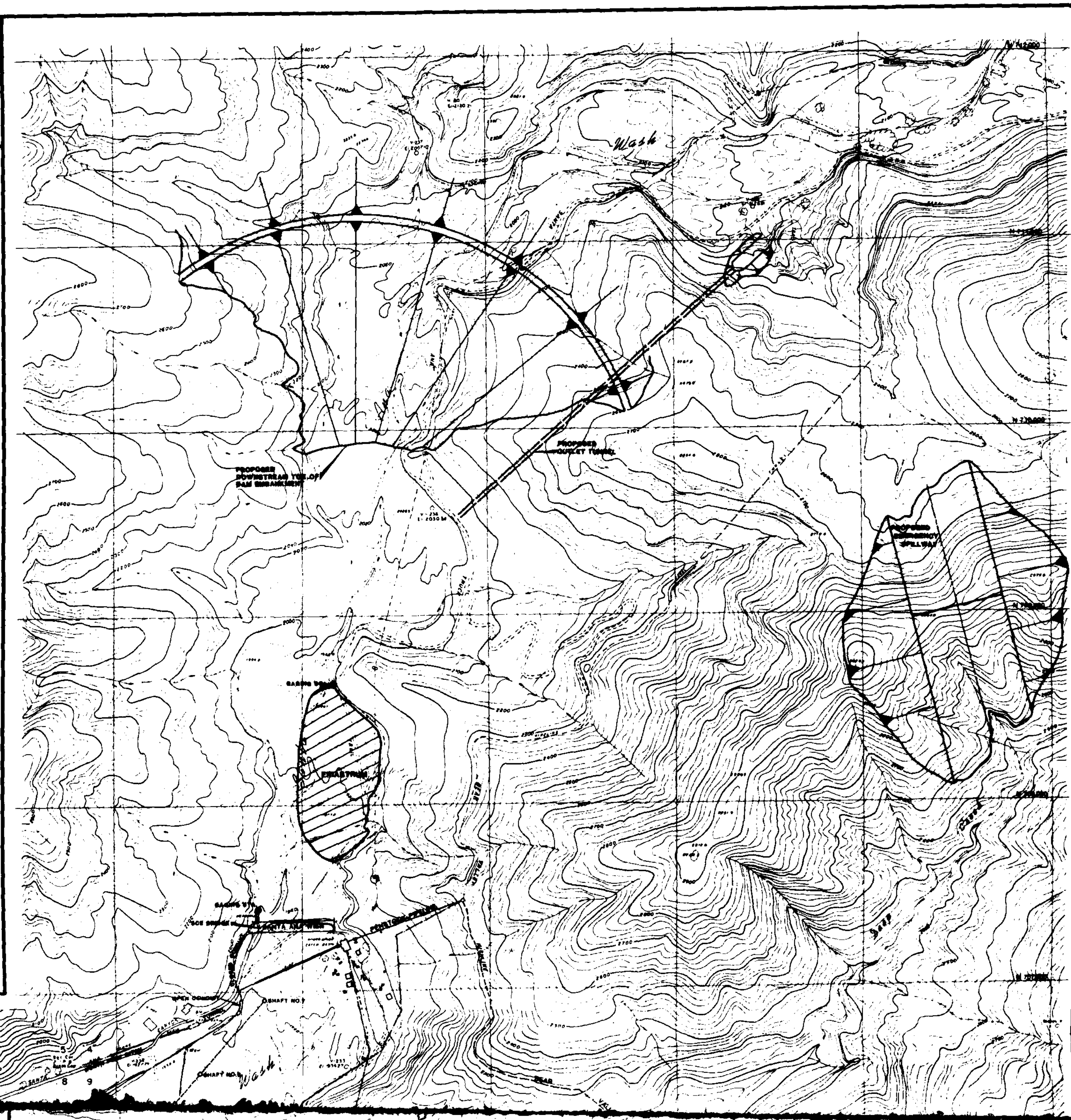
EXISTING STRUCTURES AND FACILITIES, PROPOSED DAM AND RELATED FACILITIES, AND ENDANGERED SPECIES IMPACT AREAS

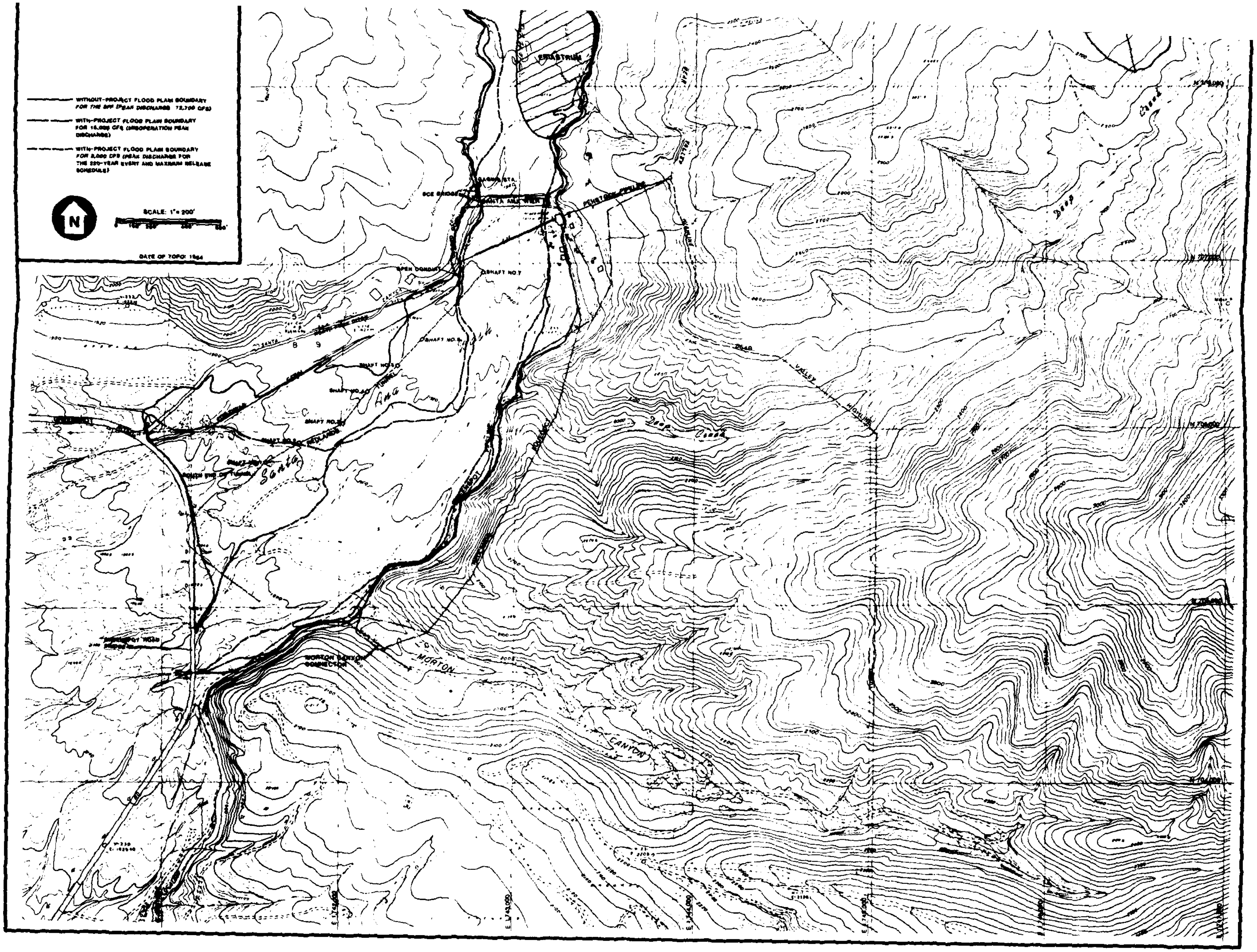
N

SCALE: 1" = 200'

0 100' 200' 400' 600'

DATE OF TOPO: 1984




WITHOUT-PROJECT FLOOD PLAIN BOUNDARY
FOR THE SPF (PEAK DISCHARGE 72,700 CFS)
WITH-PROJECT FLOOD PLAIN BOUNDARY
WITH-PROJECT FLOOD PLAIN BOUNDARY
N
SCALE: 1" = 200'
DATE OF TOPO: 1964
SCE BRIDGE
SHAFT NO 7
MORTON
CANYON
Deep Creek
Santa Ana
MORTON CANYON CONNECTOR

Simons, Li & Associates, Inc.

PROJECT NO.	CA-CAE-17
DATE:	4/87
DESIGN:	GJK
DRAWN:	TGB
CHECK:	RAM
REVISIONS:	

SHEET 2

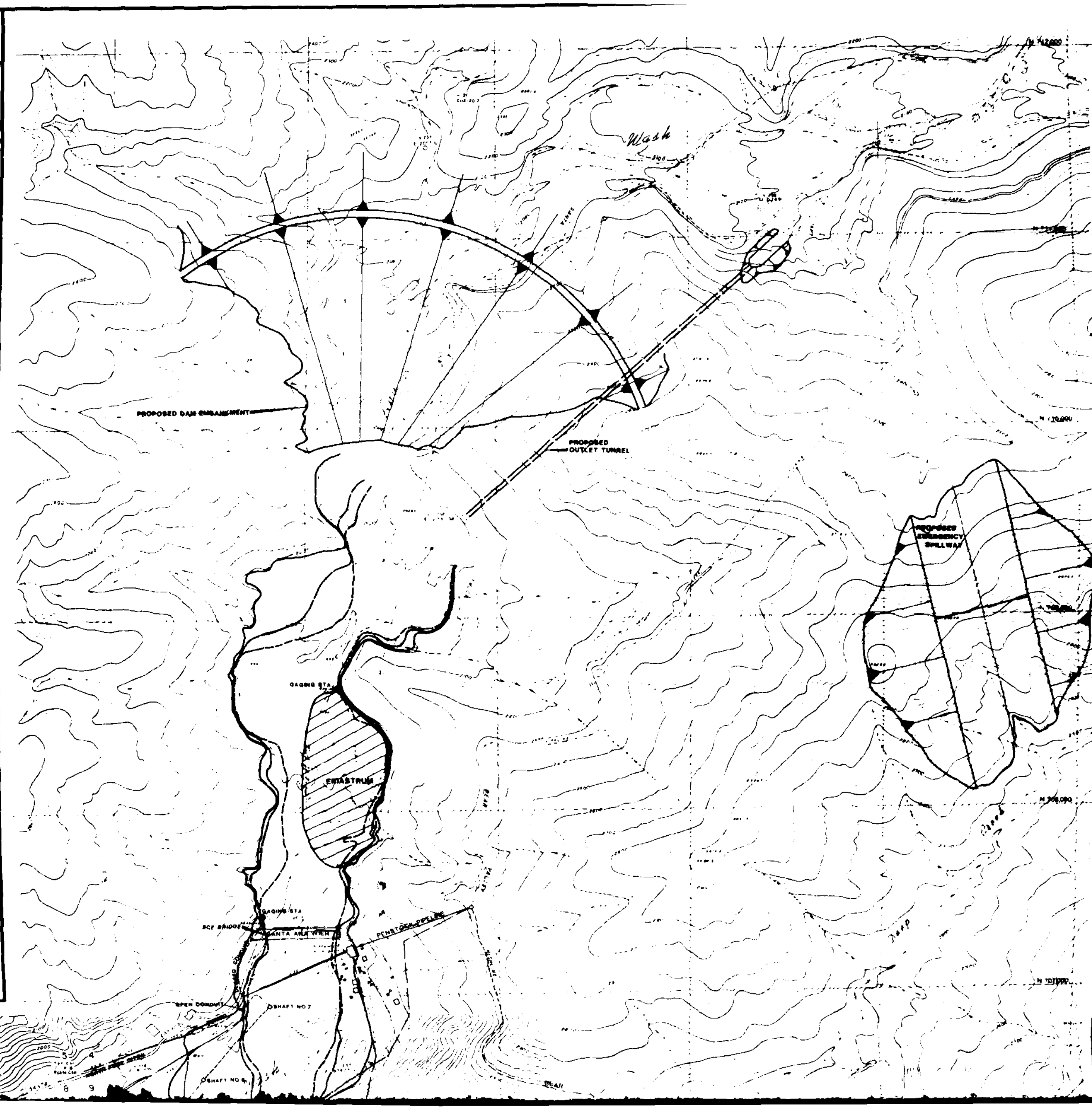
SEE TEXT PAGES 4.11 & 4.13

SANTA ANA RIVER, FLOOD CONTROL PROJECT, SEVEN OAKS DAM

LOS ANGELES DISTRICT, CORPS OF ENGINEERS

HYDRAULIC ANALYSIS

FLOOD PLAIN BOUNDARIES



WITHOUT-PROJECT FLOOD PLAIN BOUNDARY FOR THE SPF (PEAK DISCHARGE 72,700 CFS)

WITH-PROJECT FLOOD PLAIN BOUNDARY FOR 15,000 CFS (MISOPERATION PEAK DISCHARGE)

WITH-PROJECT FLOOD PLAIN BOUNDARY FOR 8,000 CFS (PEAK DISCHARGE FOR THE 220-YEAR EVENT AND MAXIMUM RELEASE SCHEDULE)

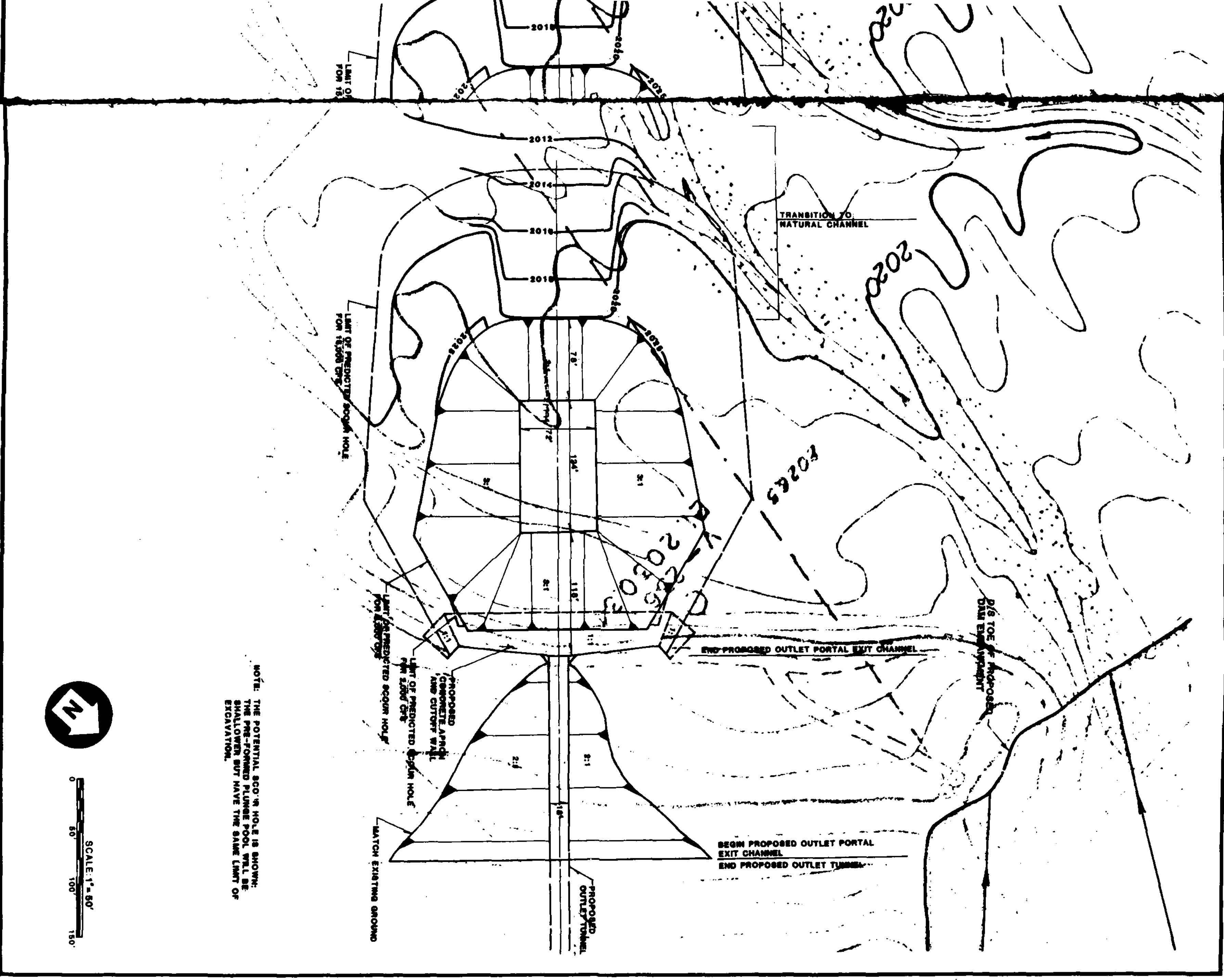
N

SCALE: 1" = 200'

0 100' 200' 400' 600'

DATE OF TOPO: 1984


2040
2020
2000
TRANSITION TO
2012
2014
ERIASTRUM



SANTA ANA RIVER, FLOOD CONTROL PROJECT, SEVEN OAKS DAM

LOS ANGELES DISTRICT, CORPS OF ENGINEERS

CONCEPTUAL DESIGN

TOE PROTECTION AND PRE-FORMED PLUNGE POOL

SIMONS, LI & ASSOCIATES, INC.

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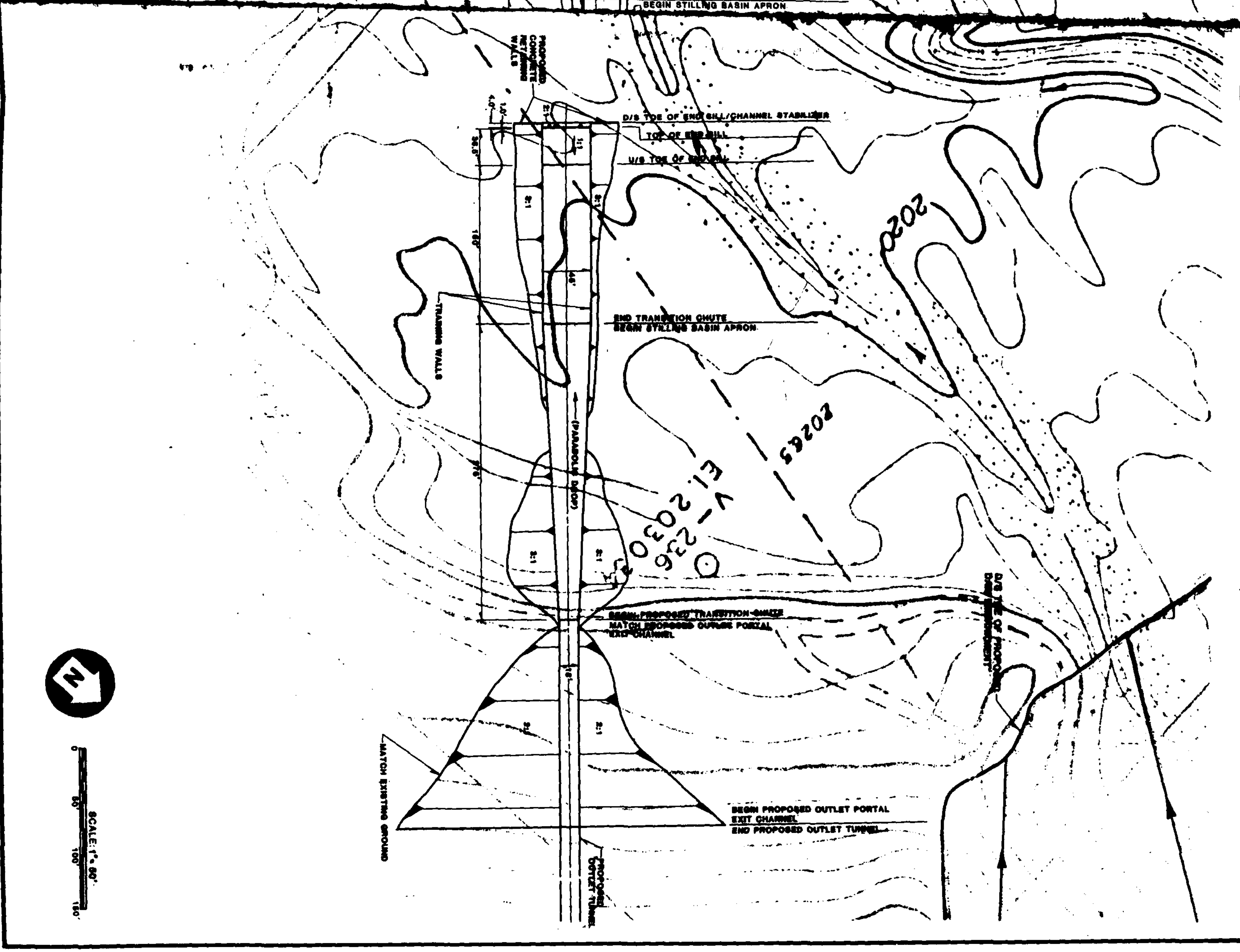
Project No	CA-COE-17
Date	4/87
Design	GJK
Drawn	TGB
Check	RAM
Revisions	

SHEET 3

SEE TEXT PAGES 4.4, 4.20 & 4.23


2040
2020
2000
PROPOSED CONCRETE RETAINING WALLS
1:1
2:1
1.0'
4.0'



Project No	CA-
Date:	4/
Design	GJ
Drawn:	T
Check:	RA
Revisions:	

SHEE

SANTA ANA RIVER, FLOOD CONTROL PROJECT, SEVEN OAKS DAM

LOS ANGELES DISTRICT, CORPS OF ENGINEERS

CONCEPTUAL DESIGN

STILLING BASIN

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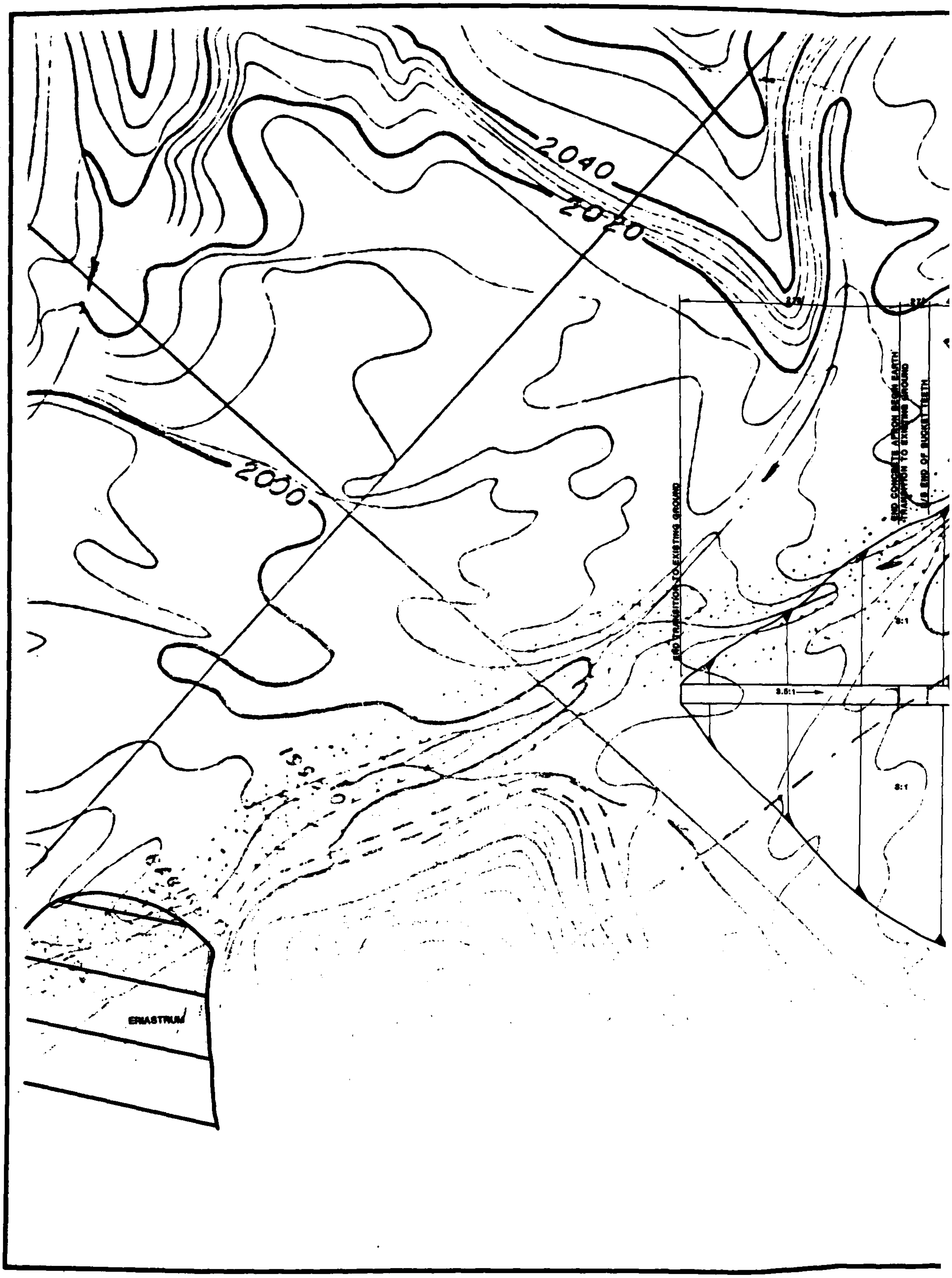
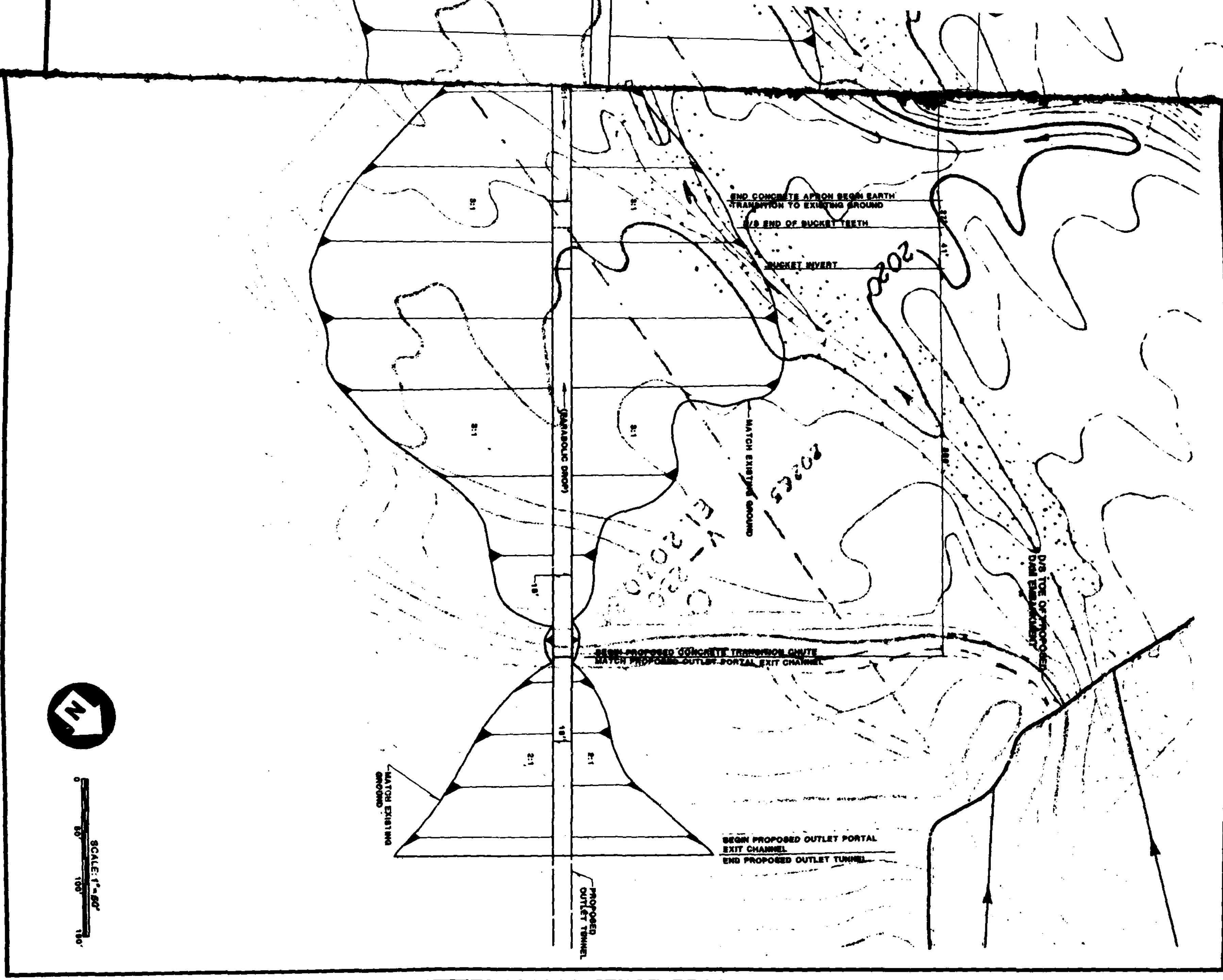
Newport Beach, CA · Phoenix, AZ · Tucson, AZ



2040
2020
2000
END TRANSITION TO EXISTING GROUND
END CONCRETE APRON BEGIN EARTH TRANSITION TO EXISTING GROUND
END OF BUCKET TEETH
3.5:1
3:1
ERIASTRUM



SANTA ANA RIVER, FLOOD CONTROL PROJECT, SEVEN OAKS DAM

LOS ANGELES DISTRICT, CORPS OF ENGINEERS

CONCEPTUAL DESIGN

SUBMERGED SLOTTED BUCKET

sla Simons, Li & Associates, Inc.
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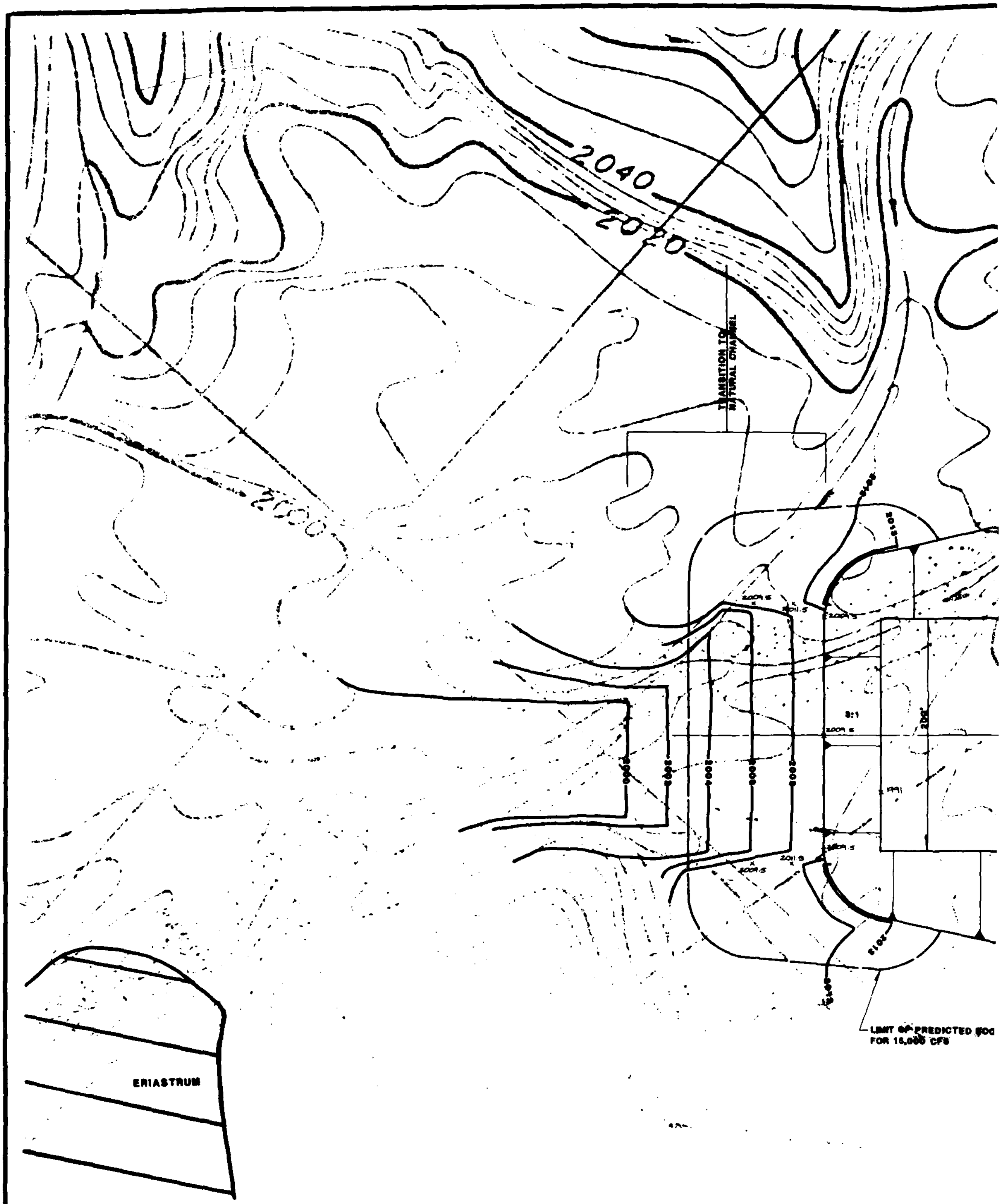
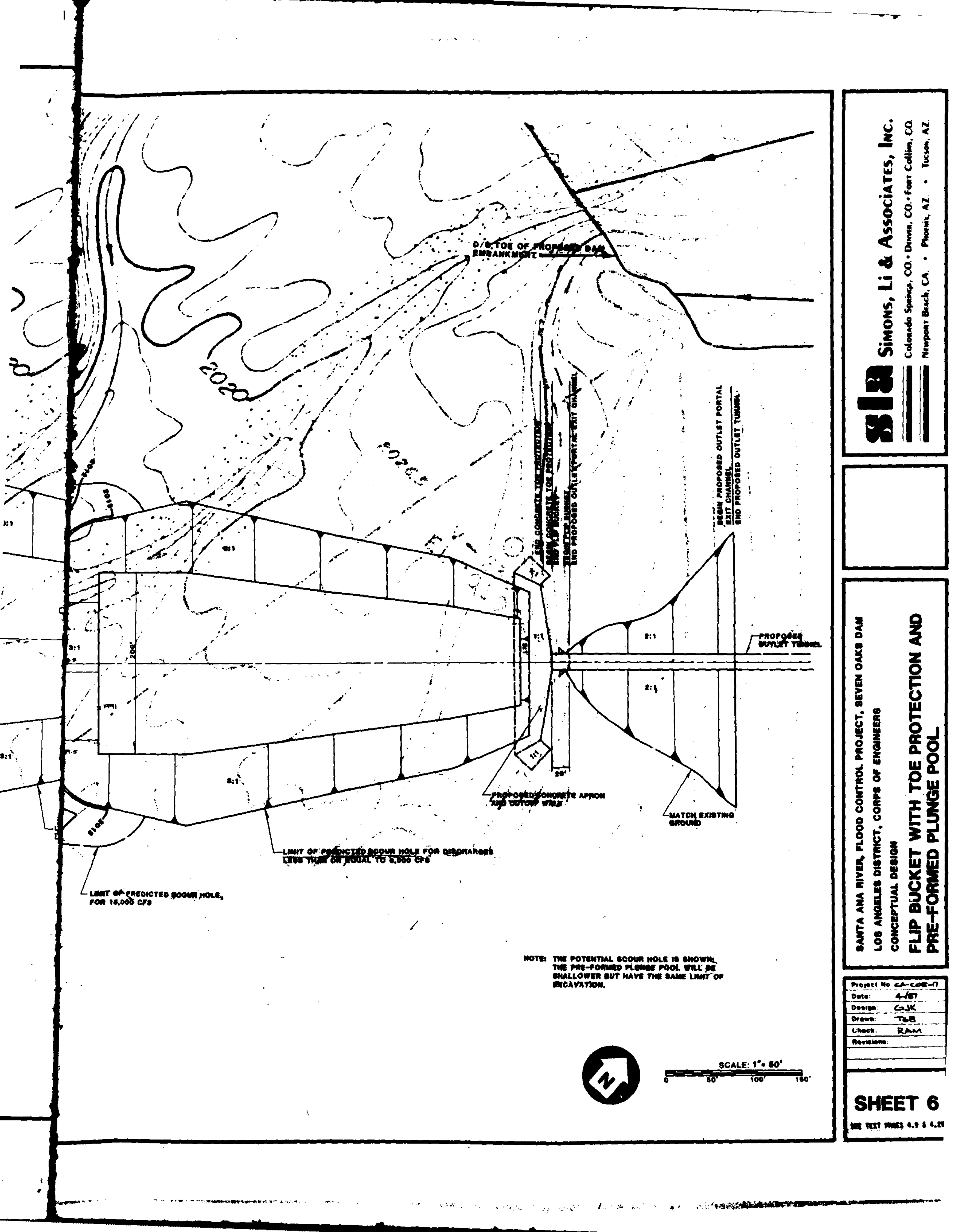
Newport Beach, CA. • Phoenix, AZ. • Tucson, AZ.

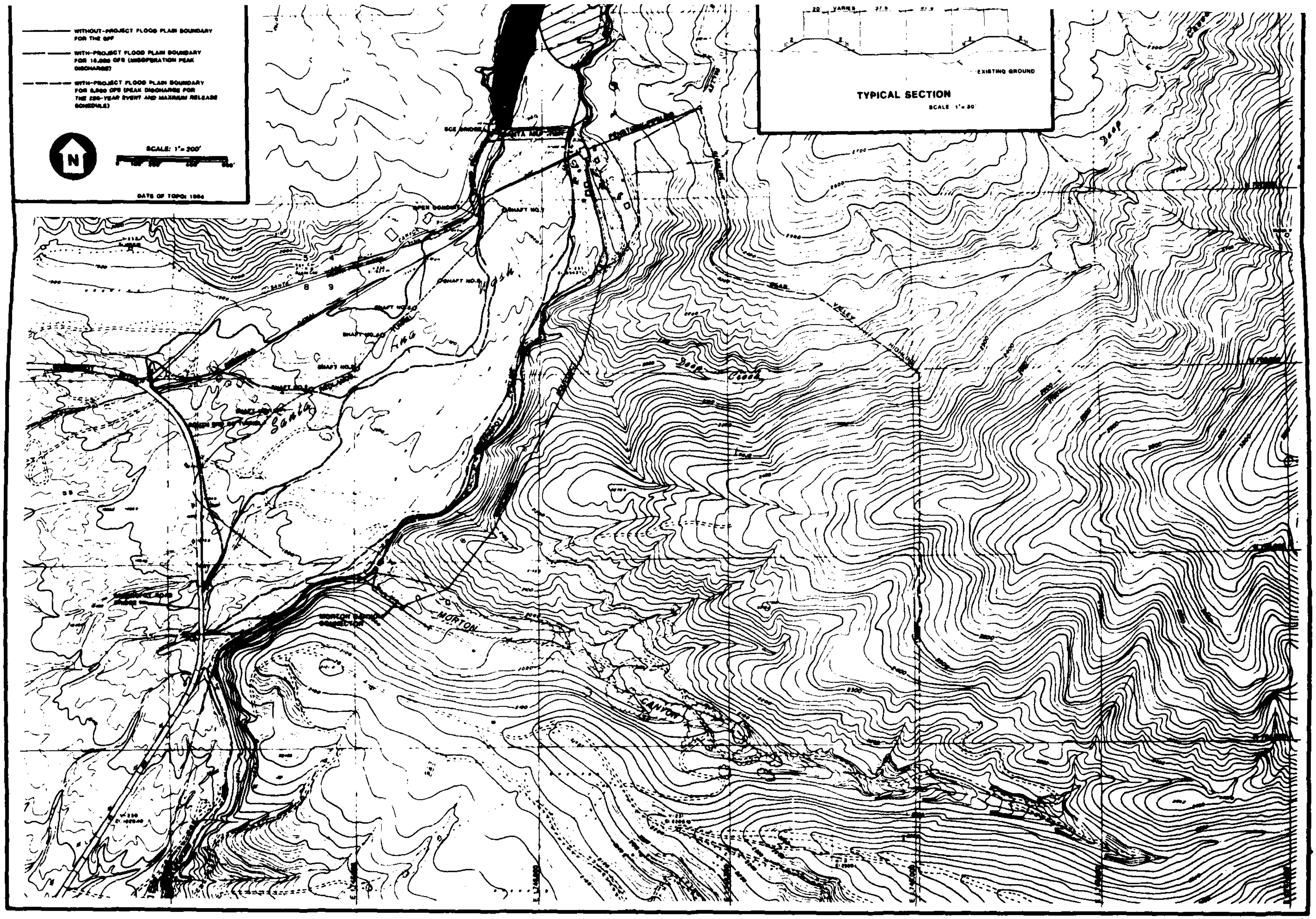
Project No CA-COE-[illegible]
Date: 4/87
Design: GJK
Drawn: TGB
Check: RAM
Revisions:

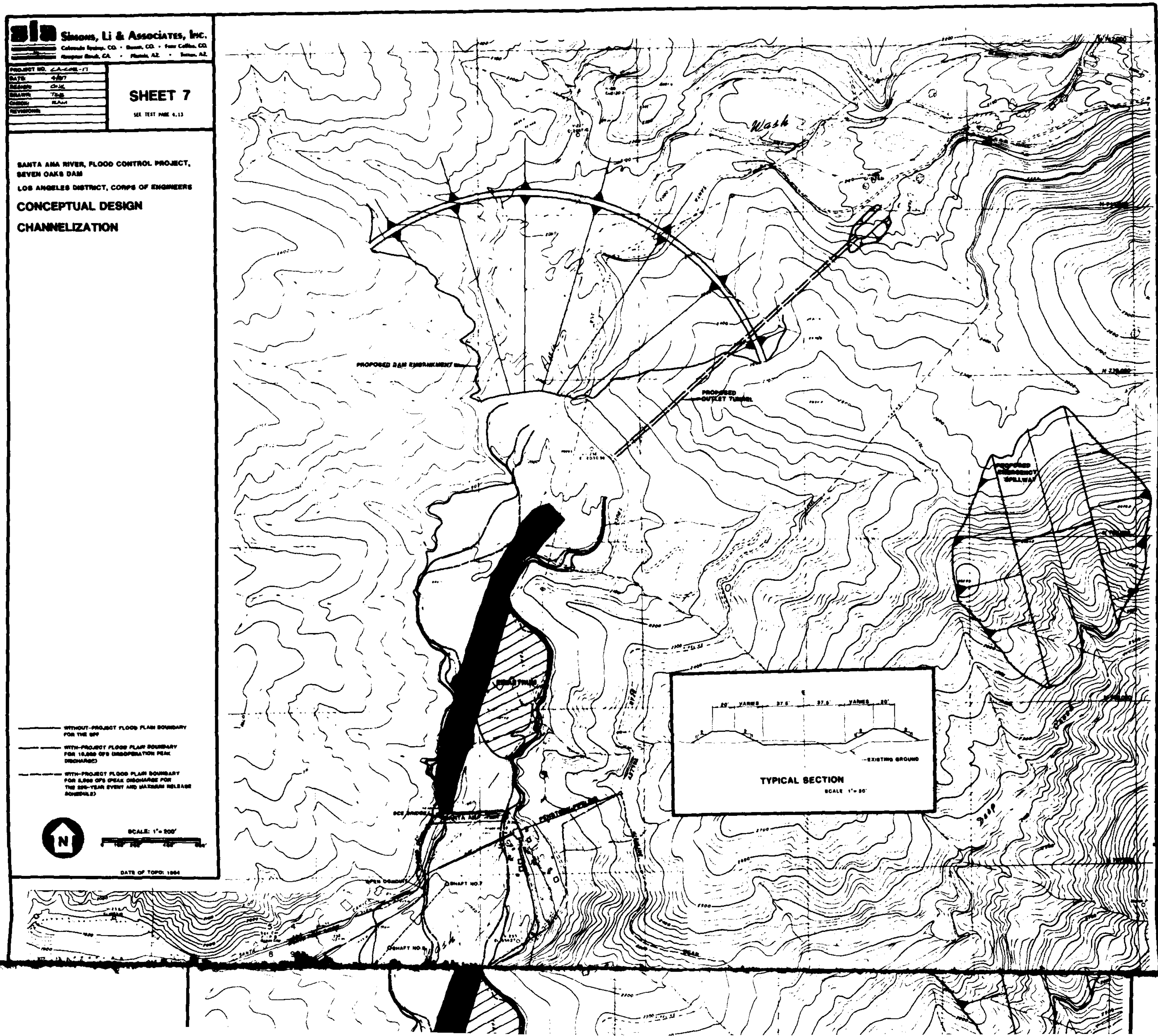
SHEET 5

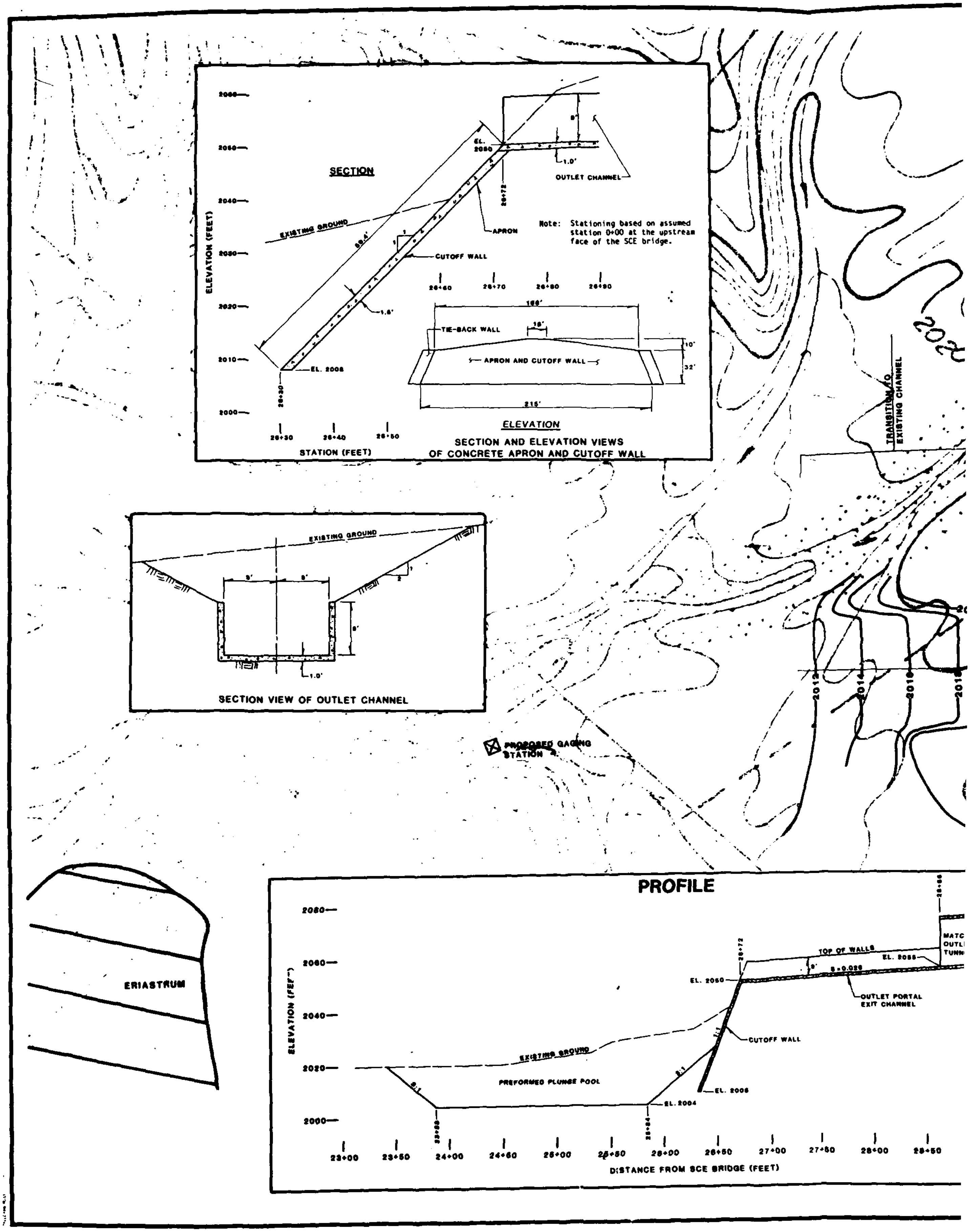
SEE TEXT PAGE 4.4

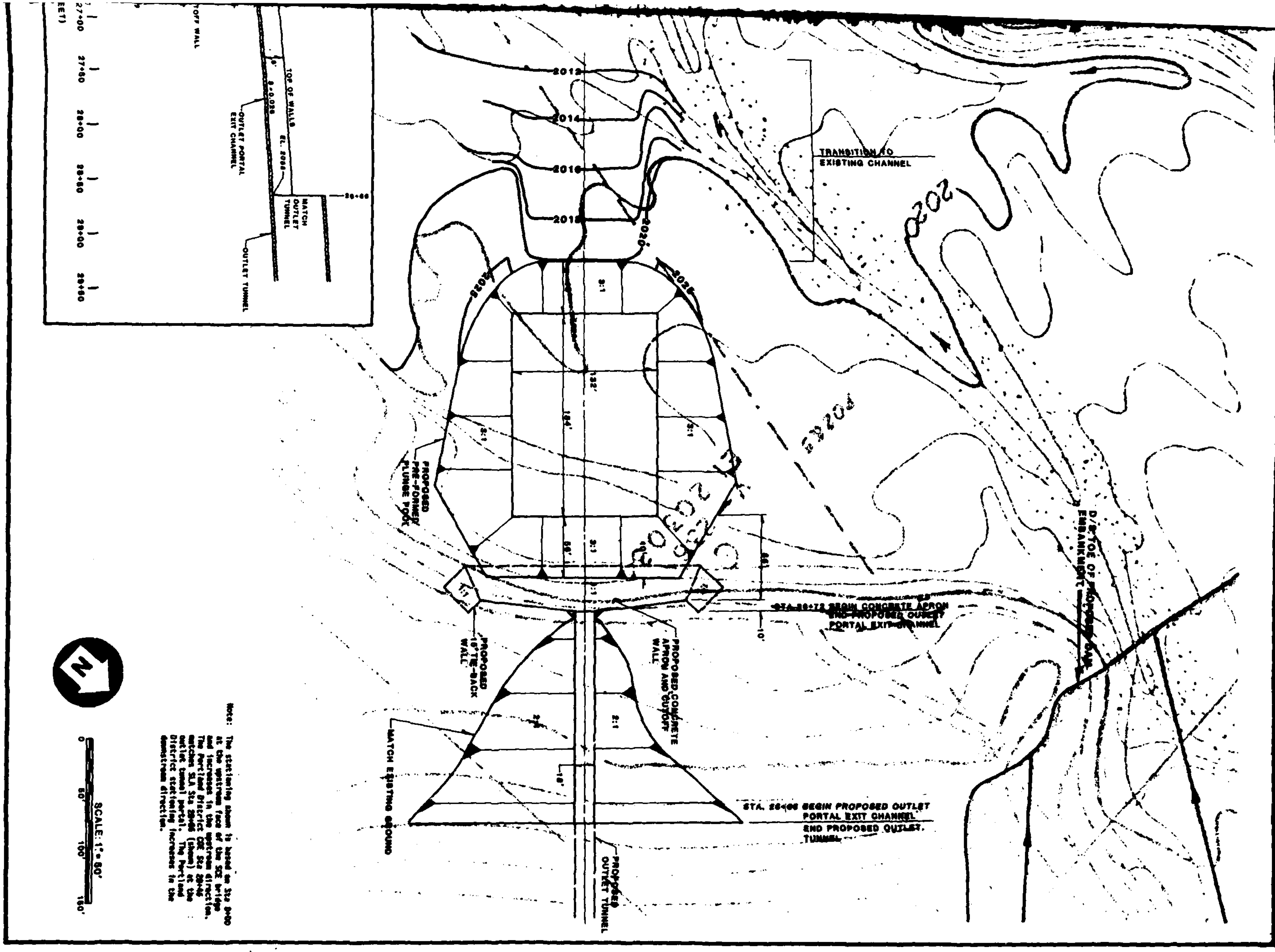

2040
2020
2000
TRANSITION TO NATURAL CHANNEL
3:1
ERIASTRUM
LIMIT OF PREDICTED
FOR 15,000 CFS


D/S TOE OF PROPOSED DAM EMBANKMENT
2020
2025
END CONCRETE TOE PROTECTION
BEGIN CONCRETE TOE PROTECTION
BEGIN PROPOSED OUTLET PORTAL
EXIT CHANNEL
END PROPOSED OUTLET TUNNEL
PROPOSED OUTLET TUNNEL
2:1
3:1
1:1
20'
PROPOSED CONCRETE APRON AND CUTOFF WALL
MATCH EXISTING GROUND
LIMIT OF PREDICTED SCOUR HOLE FOR DISCHARGES LESS THAN OR EQUAL TO 5,000 CFS
LIMIT OF PREDICTED SCOUR HOLE, FOR 16,000 CFS
NOTE: THE POTENTIAL SCOUR HOLE IS SHOWN; THE PRE-FORMED PLUNGE POOL WILL BE SHALLOWER BUT HAVE THE SAME LIMIT OF EXCAVATION.
N
SCALE: 1" = 50'
0
50'
100'
150'
sla Simons, Li & Associates, Inc.
Colorado Springs, CO. • Denver, CO. • Fort Collins, CO.
Newport Beach, CA. • Phoenix, AZ. • Tucson, AZ.
SANTA ANA RIVER, FLOOD CONTROL PROJECT, SEVEN OAKS DAM
LOS ANGELES DISTRICT, CORPS OF ENGINEERS
CONCEPTUAL DESIGN
FLIP BUCKET WITH TOE PROTECTION AND PRE-FORMED PLUNGE POOL
Project No CA-COE-17
Date: 4-/87
Design: CJK
Drawn: TSB
Check: RAM
Revisions:
SHEET 6
SEE TEXT PAGES 4.9 & 4.21


WITHOUT-PROJECT FLOOD PLAIN BOUNDARY FOR THE SPF
WITH-PROJECT FLOOD PLAIN BOUNDARY FOR 10,000 CFS (MISOPERATION PEAK DISCHARGE)
WITH-PROJECT FLOOD PLAIN BOUNDARY FOR 5,000 CFS (PEAK DISCHARGE FOR THE 230-YEAR EVENT AND MAXIMUM RELEASE SCHEDULE)
N
SCALE: 1" = 200'
DATE OF TOPO: 1984
TYPICAL SECTION
EXISTING GROUND
Santa Ana Wash
Deep Creek
MORTON
CANYON


Simons, Li & Associates, Inc.
SHEET 7
SEE TEXT PAGE 4.13
SANTA ANA RIVER, FLOOD CONTROL PROJECT, SEVEN OAKS DAM
LOS ANGELES DISTRICT, CORPS OF ENGINEERS
CONCEPTUAL DESIGN
CHANNELIZATION
WITHOUT-PROJECT FLOOD PLAIN BOUNDARY FOR THE SPF
WITH-PROJECT FLOOD PLAIN BOUNDARY FOR 10,000 CFS (MISOPERATION PEAK DISCHARGE)
WITH-PROJECT FLOOD PLAIN BOUNDARY FOR 5,000 CFS (PEAK DISCHARGE FOR THE 200-YEAR EVENT AND MAXIMUM RELEASE SCHEDULE)
N
SCALE: 1" = 200'
DATE OF TOPO: 1964
PROPOSED DAM EMBANKMENT
PROPOSED OUTLET TUNNEL
PROPOSED SPILLWAY
Wash
TYPICAL SECTION
SCALE 1" = 20'
EXISTING GROUND


SECTION
EXISTING GROUND
APRON
CUTOFF WALL
OUTLET CHANNEL
EL. 2050
EL. 2006
1.0'
1.5'
ELEVATION (FEET)
2060
2050
2040
2030
2020
2010
2000
26+30
26+40
26+50
26+60
26+70
26+80
26+90
STATION (FEET)
Note: Stationing based on assumed station 0+00 at the upstream face of the SCE bridge.
TIE-BACK WALL
APRON AND CUTOFF WALL
10'
32'
215'
ELEVATION
SECTION AND ELEVATION VIEWS
OF CONCRETE APRON AND CUTOFF WALL
EXISTING GROUND
1.0'
SECTION VIEW OF OUTLET CHANNEL
PROPOSED GAGING STATION 4
TRANSITION TO EXISTING CHANNEL
2012
2014
2016
2018
ERIASTRUM
PROFILE
TOP OF WALLS
EL. 2055
EL. 2050
OUTLET PORTAL EXIT CHANNEL
CUTOFF WALL
EXISTING GROUND
PREFORMED PLUNGE POOL
EL. 2006
EL. 2004
ELEVATION (FEET)
2080
2060
2040
2020
2000
23+00
23+50
24+00
24+50
25+00
25+50
26+00
26+50
27+00
27+50
28+00
28+50
DISTANCE FROM SCE BRIDGE (FEET)



Note: The stationing shown is based on Sta 0+00 at the upstream face of the SCE bridge and increases in the upstream direction. The Portland District COE Sta 28+46 matches SLA Sta 28+66 (shown) at the outlet tunnel portal. The Portland District stationing increases in the downstream direction.

Project No	CA-COE-17
Date:	4/87
Design:	GJK
Drawn:	TLB
Check:	RAM
Revisions:	

SHEET 8

SEE TEXT PAGES 5.1, 5.3, 5.8 & 5.1[illegible]

SANTA ANA RIVER FLOOD CONTROL PROJECT, SEVEN OAKS DAM
LOS ANGELES DISTRICT, CORPS OF ENGINEERS
PRELIMINARY DESIGN OF RECOMMENDED ALTERNATIVE

PRE-FORMED PLUNGE POOL AND TOE PROTECTION FOR OUTLET CHANNEL

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Newport Beach, CA. • Phoenix, AZ. • Tucson, AZ.